G0911-010

STRUCTURAL CALCULATIONS

FOR

Gila County Sign Shop
Russell Road and Hope Lane
Globe, AZ 85501

DLR PROJECT NO. 30-09118-00
DATE 10/30/2009

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Registered Professional Engineer (Structural)
CERTIFICATE NO. 46648
ERIC D. NEUHARTH
Date Signed 10/30/09
ARIZONA, U.S.A.
Expires 9/30/2010

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DLR Group
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Phoenix, Arizona 85016
(602.381.8580)

Project: Gila Country Sign Shop
Subject: Typical Dead Loads
Date: ########

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Roof Dead Load (For Foundations only)

Per PEMB manufacturer	5	
Collaterall	5	psf
Total	10	psf

Floor Dead Load (For Foundations only)

Per PEMB Manufacturer	75	psf
Ceiling	2	psf
Mechanical	2	psf
Electrical	1	psf
Total	80	psf

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Project: Gila Country Sign Shop
Subject: Min. Roof Live Loads - IBC 2006 Section 4.9.1
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Roof Live Load = $L_R = L_O R_1 R_2$
Where $12 <= L_R <= 20$
L_R = Reduced roof live load horiz projected area
L_O = Roof live load from Table 4-1

$R_1 = 1$, for At <=200 sq.ft.
$R_1 = 1.2 - 0.001At$ for 200 sq.ft.< At < 600 sq.ft.
$R_1 = 0.6$ for At >= 600 sq.ft.

At = Tributary area in square feet

$R_2 = 1$ for F<= 4
$R_2 = 1.2-0.05F$ for 4< F<= 12
$R_2 = 0.6$ for F>=12

F= For sloped roofs, the number inches of rise per foot
F= For arch or dome roofs, rise-to-span ratio multiplied by 32

Roof Slope	Roof Live Load Tributary Loaded Area In Square Feet For Any Structural Member				
	0 to 300	301 to 400	401 to 500	501 to 600	Over 600
	Pounds per square foot (psf)				
Flat or rise up to 4 in per foot. Arch, dome with rise-to-span ratio multiplied by 32	20	18	16	14	12
Rise 5 in per foot to less than 6 in per foot. Arch or dome with rise-to-span ratio x 32, from 4 to 6	19	17	15	13	12
Rise 6 in per foot to less than 8 in per foot. Arch or dome with rise-to-span ratio x 32, from 6 to 8	18	16	14	13	12
Rise 8 in per foot to less than 10 in per foot. Arch or dome with rise-to-span ratio x 32, from 8 to 10	16	14	13	12	12
Rise 10 in per foot to less than 12 in per foot. Arch or dome with rise-to-span ratio x 32, from 10 to 12	14	13	12	12	12
Rise 12 in per foot and greater per foot. Arch or dome with rise-to-span ratio x 32, greater than 12	12	12	12	12	12
Awnings & Canopies	5	5	5	5	5
Greenhouses	10	10	10	10	10

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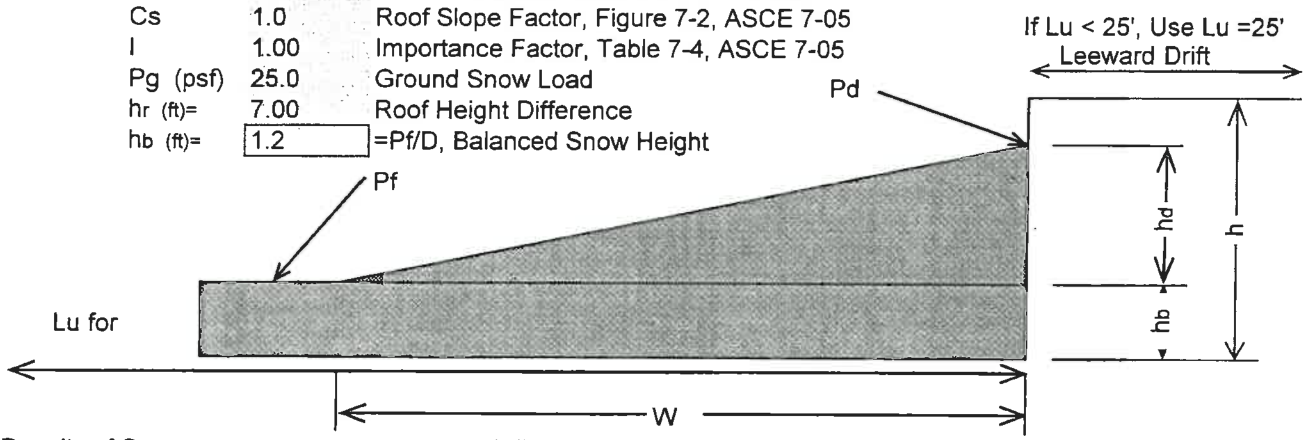
Pf = .7CeCt I Pg = 20 PSF Minimum Roof Snow Load, Pf_{MIN}=Pg*I for Pg<=20psf, Pf_{MIN}=20*I for Pg>20psf

Unbalanced snow loads shall be determined per section 7.6 ASCE 7-05

20 PSF Snow Load, for sloped roofs (Cs Pf)

40 PSF Snow Load at overhangs(2.0 Pf)

Ce	1.0	Snow Exposure Coefficient, Table 7-2, ASCE 7-05
Ct	1.0	Thermal Factor, Table 7-3 ASCE 7-02
Cs	1.0	Roof Slope Factor, Figure 7-2, ASCE 7-05
I	1.00	Importance Factor, Table 7-4, ASCE 7-05
Pg (psf)	25.0	Ground Snow Load
hr (ft)=	7.00	Roof Height Difference
hb (ft)=	1.2	=Pf/D, Balanced Snow Height



Density of Snow:
D = 0.13Pg + 14.0 <=30 (pcf)

D (pcf) = 17.25 Use: 17.25

hdl = 0.43(Lu^0.33)*(Pg + 10)^0.25 - 1.5 = Height of Leeward Drift
hdw =3/4[0.43(Lu^0.33)*(Pg + 10)^0.25 - 1.5] = Height of Windward Drift
hd=Max(hdl,hdw), If hd<=hr-hb then W = 4hd, If hd>hr-hb then hd=hr-hb & $W=4*hd^2$ /(hr-hb), but W need not exceed 8(hr-hb)
Drift Loads Need to be considered only when: (hr - hb)/hb > 0.2 or when pg > 5psf.
(Pd + Pf)max = D(hd + hb)<= D*hr

Leeward Lu (ft)	Windward Lu (ft)	Leeward hdl (ft)	Windward hdw (ft)	hd (ft)	W (ft)	Pd (psf)	Pf+Pd (psf)
25	20	1.6	1.0	1.6	6.2	27	47
40	40	2.1	1.6	2.1	8.3	36	56
50	50	2.3	1.8	2.3	9.4	41	61
60	90	2.6	2.4	2.6	10.4	45	65
80	180	3.0	3.3	3.3	13.2	57	77
100	100	3.3	2.5	3.3	13.4	58	78
120	120	3.7	2.7	3.7	14.6	63	83
140	140	3.9	2.9	3.9	15.7	68	88
160	160	4.2	3.1	4.2	16.7	72	92
180	180	4.4	3.3	4.4	17.6	76	96
200	200	4.6	3.5	4.6	18.4	79	99
250	250	5.1	3.8	5.1	20.3	88	108
300	300	5.5	4.1	5.5	22.0	95	115
350	80	5.9	2.3	5.8	23.5	101	121
400	400	6.2	4.6	5.8	26.2	101	121
450	450	6.5	4.9	5.8	28.9	101	121
500	500	6.8	5.1	5.8	31.5	101	121
550	550	7.1	5.3	5.8	34.1	101	121
600	600	7.3	5.5	5.8	36.5	101	121

For drifting snow due to parapet walls and roof projections, use Lu equal to length of upwind roof and 0.75Pd
For additional information, refer to section 7.8, ASCE 7-05, min, length of roof projection for consideration of drifting load is 15'. See section 7.9 ASCE 7-05 for sliding snow.

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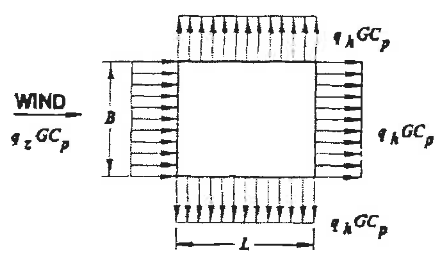
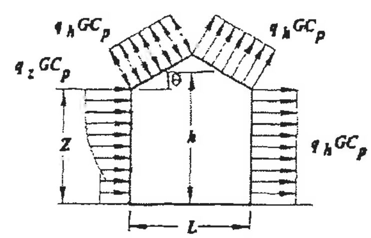
Wind Anlaysis (All Heights) - Method 2 Analytical Procedure (ASCE 7-05 , 6-5) *[ROOFS < 10 DEGREES ONLY]*

h	22.00 '		mean roof height (ft)
$h_{parapet}$	22.00 '		Top of parapet above grade (ft)
	C		Exposure Cat. & Case(B1,B2; C; D)..sec 6.5.6, ASCE 7-05
K_h	0.916		Velocity Pressure Exposure Coefficient at mean roofheight "h",.............. Table 6-3, ASCE 7-05
$K_{z\ parapet}$	0.916		Velocity Pressure Exposure Coefficient at top of parapet "hparapet",.............. Table 6-3, ASCE 7-05
K_{zt}	1.0		Topographic Factor,1.0 Default Value, Calculate for Hills, Ridges & Escarpments, Sec 6.5.7.2, ASCE7-02
K_d	0.85		Wind Directionality Factor, 0.85 for All Buildings, Table 6-4, ASCE 7-05
V	90		Basic Wind Speed (mph)............... Fig 6-1, ASCE 7-05
I_w	1.00		Importance Factor, Category I =0.87(non-hurricane), II =1.0, III =1.15, IV = 1.15, Table 6-1, ASCE 7-05
q_h	16.15		Velocity Pressure (psf) at mean roof height (h) q_h=.00256$K_hK_{zt}K_dV^2I_w$
Class	ENCL		Exposure Classification (open, par-encl, encl)..............Fig 6-5, ASCE 7-05
G	0.85		Gust Factor, 0.85 for Rigid Buildings, for Flexible Bld. see Sec 6.5.8, ASCE7-02
Length (L)	140.0 '	0.50 B/L	Length to Width $Ratio_{normal}$ (B=bldg depth in wind direction; L=bldg width transverse to wind direction)
Width (B)	70.0 '	2.00 L/B	Length to Width $Ratio_{parallel}$ (L=bldg depth in wind direction; B=bldg width transverse to wind direction)
$Area_{normal}$	4,900 sf		
$Area_{parallel}$	4,900 sf		
h/B Ratio	0.31 h/B		Height to Width $Ratio_{normal}$ (h=bldg height; B=bldg depth in wind direction)
h/L Ratio	0.16 h/L		Height to Width $Ratio_{parallel}$ (h=bldg height; L=bldg depth in wind direction)
$R_{ww\ normal}$	35.0 '		Windward $Roof_{normal}$ (horiz distance from windward edge to ridge) = B/2 if centered
$R_{ww\ parallel}$	70.0 '		Windward $Roof_{parallel}$ (horiz distance from windward edge to theoretical ridge) = L/2 if centered
Φ, Roof Slope	9.50 Deg.		Roof Slope in Degrees, Less than 10 Degrees only.





GABLE, HIP ROOF

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Main Wind Force Resisting System

	Windward	Leeward		Sidewall	Roof		
C_p	0.8	-0.50 normal	-0.30 parall	-0.7	-0.50 norml	-0.30 parall	ExternalPressure Coefficient,...... Fig 6-6, ASCE 7-05
GC_{pi}	-0.18	-0.18		0.18	0.18		Internal Pressure Coefficient,..... Fig 6-5, ASCE 7-05

$p_{total} = (q_z GC_p - q_i(GC_{pi}))_{windward} - (q_h GC_p - q_i(GC_{pi}))_{leeward}$ **(Combined Windward & Leeward)**

Design Wind Loads:

	Windward			LeeWard		LeeWard	
height (z)	K_z	q_z	p_{ww}	$p_{lw\,normal}$	p_{normal}	$p_{lw\,parallel}$	$p_{parallel}$
15 '	0.850	14.98	13.09	-3.96	**17.0**	-1.21	**14.3**
20 '	0.900	15.86	13.69	-3.96	**17.6**	-1.21	**14.9**
25 '	0.940	16.57	14.17	-3.96	**18.1**	-1.21	**15.4**
30 '	0.980	17.27	14.65	-3.96	**18.6**	-1.21	**15.9**
40 '	1.040	18.33	15.37	-3.96	**19.3**	-1.21	**16.6**
50 '	1.090	19.21	15.97	-3.96	**19.9**	-1.21	**17.2**
60 '	1.130	19.92	16.45	-3.96	**20.4**	-1.21	**17.7**
70 '	1.170	20.62	16.93	-3.96	**20.9**	-1.21	**18.1**
80 '	1.210	21.33	17.41	-3.96	**21.4**	-1.21	**18.6**
90 '	1.240	21.86	17.77	-3.96	**21.7**	-1.21	**19.0**
100 '	1.260	22.21	18.01	-3.96	**22.0**	-1.21	**19.2**
120 '	1.310	23.09	18.61	-3.96	**22.6**	-1.21	**19.8**
140 '	1.360	23.97	19.21	-3.96	**23.2**	-1.21	**20.4**
160 '	1.390	24.50	19.57	-3.96	**23.5**	-1.21	**20.8**
180 '	1.430	25.20	20.05	-3.96	**24.0**	-1.21	**21.3**
200 '	1.530	26.97	21.24	-3.96	**25.2**	-1.21	**22.5**
250 '	1.530	26.97	21.24	-3.96	**25.2**	-1.21	**22.5**
300 '	1.590	28.02	21.96	-3.96	**25.9**	-1.21	**23.2**
350 '	1.590	28.02	21.96	-3.96	**25.9**	-1.21	**23.2**
400 '	1.690	29.79	23.16	-3.96	**27.1**	-1.21	**24.4**
450 '	1.730	30.49	23.64	-3.96	**27.6**	-1.21	**24.9**
500 '	1.770	31.20	24.12	-3.96	**28.1**	-1.21	**25.3**

	+/- inter. pressure		Design Pressure
	$p_{(+)}$	$p_{(-)}$	
SideWall	-12.51	-6.70	**12.51**
$Roof_{normal}$	-9.77	-3.96	**10.00**
$Roof_{parallel}$	-7.02	-1.21	**10.00**

Top of parapet

22 '	0.916	16.15	**24.22**	**-16.15**
				40.36

Pp +/- = qp(GCpn....ASCE 7-05 6.5.12.2.4, Eq 6-20
Pww+Plw combined when diaphragm supports two ext. parpapets (ASCE 7-05 Commentary 6.5.11.5)

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COMPONENTS AND CLADDING — For A > 700 sq.ft. Use Main Wind ForceResisting System Loads

$P_{c\&c(-)} = q_h(GC_p) - q_h(GC_{pi})$ — Outward under Pos. Inter. Pressure.......ASCE 7-05, Eq 6-23

$P_{c\&c(+)} = q_z(GC_p) - q_h(-GC_{pi})$ — Inward under Neg. Inter Pressure.......ASCE 7-05, Eq 6-23

WALLS		Outward under (+) Inter. Pressure (leeward)					Inward under (-) Inter. Pressure (winward)			Design Pressure	
area	height h_z	q_h	Zone $4_{(-)}$ $GC_{p(-)}$	$p_{(-)}$	Zone $5_{(-)}$ $GC_{p(-)}$	$p_{(-)}$	q_z	Zone $4,5_{(+)}$ $GC_{p(+)}$	$p_{(+)}$	Zone 4	Zone 5
10	80	16.15	-0.90	**-17.44**	-1.800	**-31.97**	21.33	0.90	**22.10**	**22.10**	**31.97**
20	80	16.15	-0.90	**-17.44**	-1.800	**-31.97**	21.33	0.90	**22.10**	**22.10**	**31.97**
50	80	16.15	-0.85	**-16.63**	-1.560	**-28.09**	21.33	0.80	**19.97**	**19.97**	**28.09**
100	50	16.15	-0.80	**-15.82**	-1.400	**-25.51**	19.21	0.75	**17.32**	**17.32**	**25.51**
500	80	16.15	-0.70	**-14.21**	-1.000	**-19.05**	21.33	0.60	**15.70**	**15.70**	**19.05**

ROOFS		Outward under (+) Inter. Pressure						
area	height h_{mean}	q_h	Zone $1_{(-)}$ $GC_{p(-)}$	Design $p_{(-)}$	Zone $2_{(-)}$ $GC_{p(-)}$	Design $p_{(-)}$	Zone $3_{(-)}$ $GC_{p(-)}$	Design $p_{(-)}$
20	22	16.15	-1.32	**-24.22**	-2.180	**-38.10**	-3.050	**-52.15**
100	22	16.15	-1.1	**-20.67**	-1.900	**-33.58**	-2.670	**-46.01**
500	22	16.15	-0.9	**-17.44**	-1.600	**-28.74**	-2.300	**-40.04**

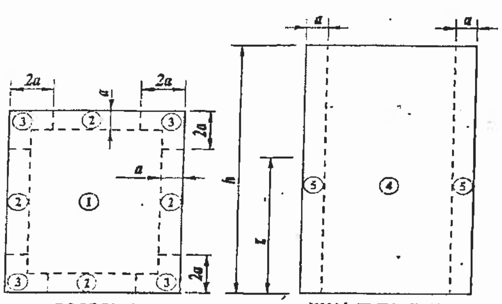
22.00' par — Parapet Height = 0.00 ft — Parapet is< 3' therefore higher pressures at Zone 3

Pc&c CASE A = qp(GCp +wall -GCp -roof); interior and corner zones....ASCE 7-05 6.5.12.4.4, Eq 6-24
Pc&c CASE B = qp(GCp +wall -GCp -wall); interior and corner zones....ASCE 7-05 6.5.12.4.4, Eq 6-24

PARAPETS			case "A" pressure towards bldg				case "B" pressure away from bldg			
area	$K_{z\,parapet}$	q_p								
20	0.916	16.15	3.08 int	**49.7 int**	3.95 crn	**63.8 crn**	-1.8 int	**-29.1 int**	-2.7	**-43.6 crn**
100	0.916	16.15	2.65 int	**42.8 int**	3.42 crn	**55.2 crn**	-1.6 int	**-25.0 int**	-2.15	**-34.7 crn**
500	0.916	16.15	2.20 int	**35.5 int**	2.90 crn	**46.8 crn**	-1.3 int	**-21.0 int**	-1.6	**-25.8 crn**

22.00' par — Parapet Height = 0.00 ft — Parapet is< 3' therefore higher pressures at Zone 3

a =	7.0 '
Roof Zone 3 (2a) =	14.0 '



ROOF PLAN — WALL ELEVATION

Notes:

1. Vertical scale denotes GC_p to be used with appropriate q_z or q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Use q_z with positive values of GC_p and q_h with negative values of GC_p.
5. Each component shall be designed for maximum positive and negative pressures.
6. Coefficients are for roofs with angle $\theta \leq 10°$. For other roof angles and geometry, use GC_p values from Fig. 6-11 and attendant q_h based on exposure defined in 6.5.6.
7. If a parapet equal to or higher than 3 ft (0.9m) is provided around the perimeter of the roof with $\theta \leq 10°$, Zone 3 shall be treated as Zone 2.
8. Notation:
   - a: 10 percent of least horizontal dimension, but not less than 3 ft (0.9 m).
   - h: Mean roof height, in feet (meters), except that eave height shall be used for $\theta \leq 10°$.
   - z: height above ground, in feet (meters).
   - θ: Angle of plane of roof from horizontal, in degrees.

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	II	OCCUPANCY CATEGORY (I, II, III & IV) TABLE 1-1 ASCE 7-05
$I_E =$	1.00	IMPORTANCE FACTOR (TABLE 11.5-1)
	C	SITE CLASSIFICATION (A, B, C, D,E,or F, TABLE 20.3-1
$S_S =$	37.4	%,MAPPED EQ SPECTRAL RESPONSE ACCELERATION, FIGURE 22-1
$S_1 =$	10.2	%,MAPPED EQ SPECTRAL RESPONSE ACCELERATION, FIGURE 22-2
$F_a =$	1.20	SITE COEFFICIENT (TABLE 11.4-1)
$F_v =$	1.70	SITE COEFFICIENT (TABLE11.4-2)
$S_{MS} =$	0.45	$S_S * F_a$, MAX. EARTH QUAKE RESPONSE ADJUSTED FOR SITE CLASS EFFECTS
$S_{M1} =$	0.17	$S_1 * Fv$, MAX. EARTH QUAKE RESPONSE ADJUSTED FOR SITE CLASS EFFECTS
$S_{DS} =$	0.30	2/3 * S_{MS}, DESIGN EARTH QUAKE SPECTRAL ACCELERATION, SHORT PERIOD
$S_{D1} =$	0.12	2/3* S_{M1}, DESIGN EARTH QUAKE SPECTRAL ACCELERATION, 1 SEC. PERIOD
	B	SEISMIC DESIGN CATEGORY, SHORT PERIOD
	B	SEISMIC DESIGN CATEGORY, 1 SECOND PERIOD
$\Omega_O =$	3	SYSTEM OVER STRENGTH FACTOR, TABLE 12.2-1
		Per PEMB Manufacuter, likely not detailed for seismic resistance
$R =$	3	RESPONSE MODIFICATION FACTOR FROM TABLE12.2-1
$C_S =$	0.100	$S_{DS}/(R/I_E)$, $C_{SMAX} = S_{D1}/(R/I_E)T$ for $T <= T_L$ OR
		$C_{SMAX} = S_{D1}*T_L/(R/I_E)T^2$ for $T > TL$
$T_L =$	6	Long- period transition period Per ASCE 7-05 Figure 22-15 to 22-20
$T =$	0.246	Fundamental period determined by analysis, or Default Value of $T < Cu\ Ta$
C_S min ($S_{1M} > 0.6$) =	0.010	$0.5*S_1/(R/I_E)$ WHERE $S_{M1} > 0.6g$.
$h_n =$	14	HEIGHT IN FEET TO UPPER MOST LEVEL FROM BASE
$T_a =$	0.145	APPROXIMATE PERIOD OF VIBRATION OF THE BUILDING (SECONDS) $= C_t h_n^x$
$C_t =$	0.02	VIBRATION PERIOD COEFFICIENT (= 0.028 for steel MRF, 0.016 for concrete MRF, 0.03 for eccentrically braced steel frames, 0.02 for all other structural systems)
$x =$	0.75	VIBRATION PERIOD EXPONENT (= 0.8 for SMRF, 0.9 for CMRF, 0.75 for all other)
$W =$		TOTAL WEIGHT OF STRUCTURE

$C_{SMIN} =$	0.010
$C_{SMAX} =$	0.156
$S_{DS}/(R/I_E) =$	0.100

DESIGN FORMULA	BASE SHEAR	REMARKS
$V = C_S * W$ =	0.100 * W	

$E = \rho*Q_E +- 0.2S_{DS}D$ E= SEISMIC EFFECT FOR USE IN LOAD COMBINATIONS

$E_m = \Omega_o*Q_E +- 0.2S_{DS}D$ $E_m =$ SEISMIC EFFECT FOR USE IN SPECIAL LOAD COMBINATIONS

$Q_E =$ EFFECT OF HORIZONTAL SEISMIC FORCES, V or Fp

D= EFFECT OF DEAD LOAD FORCES

$\rho =$ RELIABILITY FACTOR = 1.00

USE ρ =1.3 FOR SDC= D, E or F UNLESS ONE OF TWE CONDITIONS ARE MET, PER 12.3.4.2

= 1 FOR SEISMIC DESIGN CATEGORIES A,B,& C

E=	1.00	* Q_E +-	0.06	*D
E_m=	3	* Q_E +-	0.06	*D

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STRUCTURAL COMPONENTS (PART OF SEISMIC FORCE-RESISTING SYSTEM):

Collectors Elements in Seismic Design Categories C through F : Section 12.10.2.1

Collector elements splices and their connections shall be designed with load combinations with overstrength.

Structural walls and their Anchorage: Section 12.11, (All Seismic Design Categories)

Perpendicular to wall loads:

$F_P = 0.4\ I_E\ S_{DS}\ W_W$, or $F_p = 0.10\ W_W = \boxed{0.120}\ W_W$ $\quad W_w$ = WEIGHT OF WALL

Anchorage of walls: F_P ABOVE OR 400 $S_{DS}\ I_E$ >= 280 PLF

$400\ S_{DS}\ I_E = \boxed{120}$ PLF >= 280 PLF

Seismic Design Category C and above : In accord with Category B except as follows:

Anchorage of walls: Out-of-Plane Wall Anchorage to Flexible Diaphragms:

$= 0.8\ S_{DS}\ I_E\ W_p = \boxed{0.239}$ Wp , PLF >= 280 PLF

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NON-STRUCTURAL COMPONENTS:

$F_p = 0.40*a_p*S_{DS}*I_p*W_p/(R_p)*(1+2*z/h)$

$F_p >= .3*S_{DS}*I_p*W_p =$ $\boxed{0.08976}$ I_p*W_p

$F_p <= 1.6*S_{DS}*I_p*W_p =$ $\boxed{0.479}$ I_p*W_p

I_p = COMPONENT IMPORTANCE FACTOR (SECTION 13.1.3)

z = HEIGHT OF COMPONENT (FT), z<= h

h = 17 AVERAGE ROOF HEIGHT (FT)

*Per Section 9.6.1.6.1, R_p = 1.5 for shallow anchors (embedment length/diameter < 8)

**Per Section 13.4.2, increase design load by 1.3 for anchors embedded in concrete or masonry

DESCRIPTION OF ELEMENT (FROM TABLES 13.5-1 OR 13.6-1	a_p	R_p*	z	Ip**	F_p	REMARKS
INTERIOR NON-STRUCTURAL UNREINFORCED MAS. PARTITIONS	1.0	1.5	14	1.0	0.211 $*W_p$	
ALL OTHER INTERIOR NON-STRUCTURAL PARTITION WALLS	1.0	2.5	14	1.0	0.127 $*W_p$	
ALL OTHER INTERIOR NON-STRUCTURAL PARTITION WALLS	1	2.5	14	1.0	0.127 $*W_p$	
EXTERIOR NON-STRUCTURAL WALLS AND CONNECTIONS						
WALL ELEMENT	1	2.5	14	1.0	0.127 $*W_p$	
BODY OF CONNECTION	1	2.5	14	1.0	0.127 $*W_p$	
CONNECTION FASTENERS	1.25	1.0	14	1.0	0.396 $*W_p$	
CANTILEVERED PARAPETS	2.5	2.5	14	1.0	0.317 $*W_p$	
INTERIOR NON-STRUCTURAL UNREINFORCED MAS. PARTITIONS	1.0	1.5	20	1.0	0.268 $*W_p$	
ALL OTHER INTERIOR NON-STRUCTURAL PARTITION WALLS	1.0	2.5	20	1.0	0.161 $*W_p$	
ALL OTHER INTERIOR NON-STRUCTURAL PARTITION WALLS	1	2.5	20	1.0	0.161 $*W_p$	
EXTERIOR NON-STRUCTURAL WALLS AND CONNECTIONS						
WALL ELEMENT	1	2.5	20	1.0	0.161 $*W_p$	
BODY OF CONNECTION	1	2.5	20	1.0	0.161 $*W_p$	
CONNECTION FASTENERS	1.25	1.0	20	1.0	0.479 $*W_p$	
					#DIV/0! $*W_p$	
					#DIV/0! $*W_p$	
					#DIV/0! $*W_p$	

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Project Gila Country Sign Shop
Subject: Concrete Reinforcing Splice Lengths
Date: ######

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Reinforcing Splice Length Table Per ACI 318-05 and 2006 IBC - (Inches)												
Rebar Size				Concrete member								
In-LB	Soft Metric	Area (in^2)	Diam. (in)	Footing	Grade Beam (top)	Grade Beam (bott.)	Wall Horiz. (top)	Wall Vert.	Slab	Beam (top)	Beam (bott.)	Column
#3	#10	0.11	0.375	16	17	16	12	16	16	16	16	*
#4	#13	0.20	0.500	19	23	18	19	19	20	20	16	15
#5	#16	0.31	0.625	24	28	22	28	28	30	25	19	19
#6	#19	0.44	0.750	29	34	26	37	37	40	29	23	23
#7	#22	0.60	0.875	41	49	38	60	60	64	48	37	27
#8	#25	0.79	1.000	47	56	43	74	74	80	61	47	30
#9	#29	1.00	1.128	53	69	54	90	90	96	75	58	34
#10	#32	1.27	1.270	60	85	66	108	108	116	91	70	39
#11	#36	1.56	1.410	66	103	79	127	127	136	109	84	43
#14	#43	2.25	1.690	*	*	*	*	*	*	*	*	*
#18	#57	4.00	2.257	*	*	*	*	*	*	*	*	*

* Generally not permitted

Design Variables

	Footings	Grade Beam		Wall		Slab	Beam		Columns
		Top	Bottom	Horiz.	Vert.		Top	Bottom	
fy (psi)	60000	60000	60000	60000	60000	60000	60000	60000	60000
f'c (psi)	2500	3000	3000	4000	4000	3500	4000	4000	3000
ψ_t	1	1.3	1	1.3	1	1	1.3	1	
ψ_e	1	1	1	1	1	1	1	1	
ψ_s(3–6)	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
ψ_s(7+)	1	1	1	1	1	1	1	1	
λ	1	1	1	1	1	1	1	1	
Clear Cover c_b	3	2	2	0.75	0.75	0.75	1.5	1.5	
Bar spacing	7.5	5.5	5.5	3	3	12	4.5	4.5	
Ktr	0	0	0	0	0	0	0	0	
Class	1.3	1.3	1.3	1	1.3	1.3	1.3	1.3	1.[illegible]

ACI Section 12.2.3, Eqn. (12-1): l_d * Class= $\dfrac{3 * f_y * \psi t \psi e \psi s \lambda * d_b * CLASS}{40*(f'c)^{1/2}\ ((c_b+K_{tr})/d_b)}$ = Splice length (see table)

$(c_b+K_{tr})/db$ limited to <= 2.5

ψ_t – Reinforing location factor (= 1.3 for top, = 1.0 for bottom)

ψ_e – Coating factor = 1.0 for no coating (increase to 1.2 or 1.5 for epoxy coating). See section 12.2.4.
Note: $\psi_t * \psi_e$ can be limited to a maximum value of 1.7, but is not in this spreadsheet.

ψ_s – Reinforcing size factor (0.8 up to #6, 1.0 for #7 and up)

λ – Aggregate Factor (= 1.0 for normal weight, =1.3 for lightweight)

c_b - Cover factor = the lesser of:
smallest clear cover to any edge + $d_b/2$, or one-half the center to center spacing of bars.

K_{tr} - Transverse reinf. index . A reduction factor if the bars being spliced are enclosed within stirrups. It is conservative to consider Ktr = 0. See section 12.2.4.

Class - = 1.0 for Class A or = 1.3 for Class B tension lap splices. Use Class B as typical unless you are sure that the bars will be less than 50% stressed in flexure and half or less of reinforcing is spliced within the required lap length.

SPLICES OFCOMPRESSION REINFORCING: (ACI 318-05 12.16)

l_{dc}= 0.0005 * $f_y \cdot d_b$ for f_y =< 60,000 PSI, or (0.0009 * f_y - 24) * d_b for f_y > 60,000 PSI
12" MINIMUM, IF f 'c < 3000 PSI than increase length of lap by 1/3

Project: Gila Country Sign Shop
Subject: CONCRETE BEAM DESIGN
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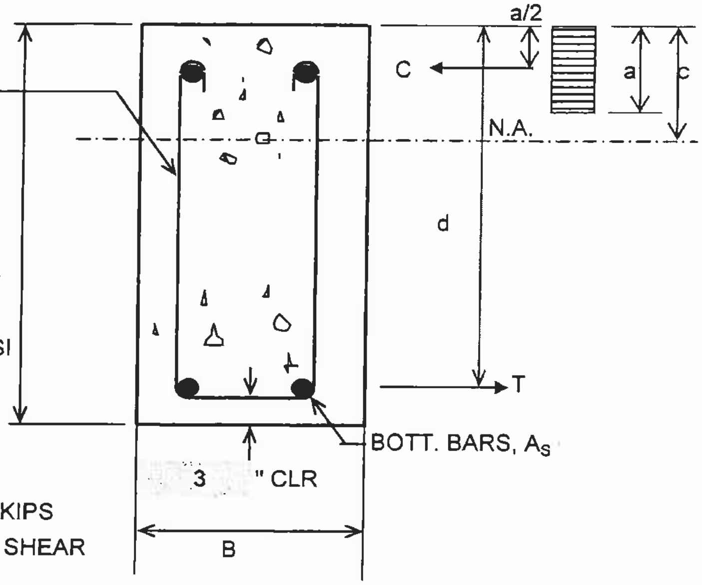
GENERAL DESIGN INFORMATION:



B = BEAM WIDTH, IN.
H = BEAM HEIGHT, IN.
d = DEPTH TO REINF. STEEL, IN.
= H - CLR - .375" - .5"
A_S = AREA OF BOTT. STEEL, IN^2.
A_V = AREA OF SHEAR REINF., IN^2.

f'_C = SPECIFIED COMPRESSIVE STRENGTH OF CONCRETE, PSI.
f_Y = SPECIFIED YIELD STRENGTH OF REINF.,PSI
Φ = STRENGTH REDUCTION FACTOR = 0.90
a = DEPTH OF EQUIVALENT RECTANGULAR BLOCK = β_1*c
M_N = NOMINAL MOMENT STRENGTH, KIP-FEET
V_C = NOMINAL CONCRETE SHEAR STRENGTH, KIPS
V_S = NOMINAL SHEAR STRENGTH PROVIDE BY SHEAR REINFORCEMENT, KIPS
V_N = NOMINAL SHEAR STRENGTH, KIPS = Vc + Vs
β_1 = 0.85 FOR f 'c <= 4000 PSI OR
= 0.85 - 0.05 x (f'_C -4000) / 1000
S = SPACING OF SHEAR REINF.
c = DISTANCE TO NEUTRAL AXIS,IN.
E_S = MODULAS OF ELASTICITY OF REINF., 29,000 KSI
E_C = MODULAS OF ELASTICITY OF CONCRETE,
= 57,000 $\sqrt{f'_C}$/1000, KSI
n = MODULAR RATIO, E_S / E_C

BENDING: C = T $\quad$ $C = 0.85 * f'_C * B * a$ $\quad$ $T = A_S * f_Y$ $\quad$ $a = A_S * f_Y / (0.85 * f'_C * B)$

$\Phi M_N = \Phi\,[A_S * f_Y\,(d - a/2) * 1/12000\,]$ $\quad$ $\Phi = 0.9$ $\quad$ $\rho_{MIN} = 200 / f_Y$

$\rho_{BAL} = 0.85\, f'_C\, \beta_1 / f_Y * [87{,}000 / (87000 + f_Y)]$

SHEAR: $V_C = 2\sqrt{f'_C}\; B * d$

$V_S = A_V * f_Y * d / S$ $\quad$ $S_{MAX} = d/2$ $\quad$ $V_{SMAX} = 8\sqrt{f'_C}\; B * d$

$\Phi V_N = \Phi V_C + \Phi V_S$ $\quad$ $\Phi = 0.75$

DEFLECTION: I_{CR} = MOMENT OF INERTIA OF TRANSFORMED CRACKED SECTION

$I_{CR} = 1/3 * B * c^2 + n * A_S * (d - c)^2$

SOLVE FOR X : $c^2 + 2n\,A_S/B\;c - 2nA_S\,d/B = 0$

$$c = \frac{-(2nA_S/B) \pm \sqrt{(2nA_S/B)^2 + 4(2nA_S d/B)}}{2}$$

Project: Gila Country Sign Shop
Subject: CONCRETE BEAM DESIGN
Date: 10/22/09

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TABLE OF CONCRETE BEAM CAPACITIES:

f'_c = 3,000 PSI
β_1 = 0.85

f_Y = 60,000 PSI
n = 9.3

ρ_{MIN} = 0.0033
$\rho_{MAX} = 0.75\rho_{BAL}$ = 0.0160

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MARK	B	H	BOTT. BARS, A_S		SHEAR REINF, A_V		ΦM_N	ΦV_N	I_{CR}
	IN.	IN.	NO.	SIZE	SIZE	SPACING	FT*K	K	IN.^{4}
GB-1	24	24	4	6	4	48	153.1	39.7	4123
GB-2	36	24	AS< ASmin=2.415 5	6	4	48	192.9	59.5	5386
GB-3	48	24	AS< ASmin=3.22 6	6	4	48	232.3	79.4	6618
GB-4	24	30	AS< ASmin=2.09 4	6	4	48	200.9	51.5	7324
GB-5	36	30	AS< ASmin=3.135 5	6	4	48	252.5	77.3	9519
GB-6	48	30	AS< ASmin=4.18 6	6	4	48	303.9	103.0	11665
GB-1	24	36	AS< ASmin=2.57 4	7	4	48	335.0	63.3	14639
GB-1	36	36	AS< ASmin=3.855 5	7	4	12	421.3	142.3	19068
GB-1	48	36	AS< ASmin=5.14 6	7	4	48	507.2	126.7	23394
GB-1	24	48	AS< ASmin=3.53 4	7	4	48	464.8	87.0	29402
GB-1	36	48	5	7	4	48	583.7	130.5	38073
GB-1	48	48	AS< ASmin=7.06 6	7	4	48	702.0	174.0	46563

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MINIMUM BEAM WIDTH (INCHES) ACCORDING TO THE ACI CODE

B = BEAM WIDTH (IN.)

$B = 2\ CLR + 2\ D_V + 2(0.293\ (0.75 - 0.5D_B)) + ND_B + (N-1)\ [D_B \text{ OR } 1\text{"MIN}]$

D_B = BAR DIAMETER (IN.)

CLR = SIDE CLEAR DISTANCE (IN.)

D_V = STIRRUP BAR DIAMETER (IN.)

N = NUMBER OF BARS

CLR = 3

D_V = 0.375

MINIUM BEAM WIDTH (B, INCHES)									
SIZE OF BAR	NUMBER OF BARS IN SINGLE LAYER OF REINFORCEMENT (N)								ADD FOR EACH ADDED BAR
	2	3	4	5	6	7	8	9	
4	9.0	10.5	12.0	13.5	15.0	16.5	18.0	19.5	1.5
5	9.3	10.9	12.5	14.1	15.8	17.4	19.0	20.6	1.625
6	9.5	11.2	13.0	14.7	16.5	18.2	20.0	21.7	1.75
7	9.7	11.6	13.4	15.3	17.2	19.1	20.9	22.8	1.875
8	9.9	11.9	13.9	15.9	17.9	19.9	21.9	23.9	2.00
9	10.2	12.5	14.7	17.0	19.2	21.5	23.7	26.0	2.25
10	10.6	13.1	15.6	18.1	20.6	23.1	25.6	28.1	2.50
11	10.9	13.7	16.4	19.2	21.9	24.7	27.4	30.2	2.75

Project:	Gila Country Sign Shop	Project Number:	30-09118-00
Subject:	ANCHOR DESIGN ACI 318-05 APPENDIX D	Computed by:	EDN
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GENERAL INPUT

Loads are calculated for Load Combinations of ACI 318 Section 9.2 , Thus this spread sheet uses the strength reduction factors of section D4.4

This spread sheet does not apply to adhesive anchors, or grouted anchors.

When anchors design includes seismic loads, additional requirements of D.3.3.3 -D.3.3.5 shall be applied. (Apply an additional strength reduction factor of 0.75 for anchors resisting moderate or high seismic loads.)

Anchor Bolts Per ASTM F 1554:

Grade	Tensile Strength (KSI)	Yield Strength (KSI)	Size Range	
36	58 - 80	36	1/4" - 2"	Ductile
55	75- -95	55	1/4" - 2"	Ductile
105	125-150	105	1/4" - 2"	Ductile

Per Standard See Table 2, and definition of Ductile steel element per ACI 318 Appendix D

Anchor Bolts Per ASTM A307, A325, A490, Table 7-10 & Table 7-14 AISC 3rd Edition

ASTM	ΦFv (kips)	ΦFt	
A307	18	33.8	Ductile
A325	36	67.5	
A490	45	84.8	

Threads included in shear plane

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Project: Gila Country Sign Shop
Subject: Spread Footing Design
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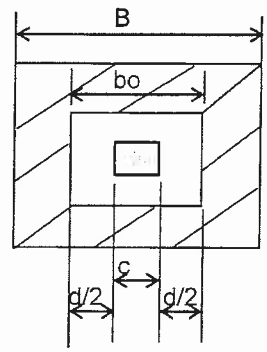
SPREAD FOOTING DESIGN: PER ACI 318-05 and 2006 IBC

Pa = SPREAD FOOTING CAPACITY, KIPS
f 'c = CONCRETE COMPRESSIVE STRENGTH AT 28 DAYS, PSI
Fy = REINFORCING STEEL YIELD STRENGTH, PSI
Qa = SAFE NET ALLOWABLE SOIL BEARING PRESSURE, PSF
B = WIDTH OF FOOTING, FT.
A = BEARING AREA OF FOOTING, BxB, SQ.FT.
c = MINIUM COLUMN DIMENSION, IN.
h = DEPTH OF FOOTING,IN.
CLR = CLR DISTANCE, 3"
d = DEPTH TO REINFORCEMENT, d = h - CLR. - 1/2", IN.
LF = LOAD FACTOR FOR CONCRETE DESIGN
Qu = FOOTING BEARING PRESSURE UNDER OVERLOAD, Qa x LF, PSF

FOOTING CAPACITY BASED ON SOIL BEARING PRESSURE:

Pa = A x Qa



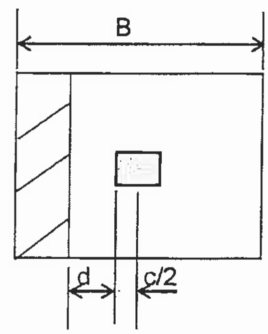
TWO- WAY SHEAR: CRITICAL SECTION d/2 FROM FACE OF COLUMN

DETERMINE d REQUIRED FOR Pa ABOVE.
vc = 4 x SQRT(f 'c)

Vu = Qu (A - (bo/12)^2)
ΦVc = Φ * vc *4*bo* d
Vu = Φ Vc

bo = c + d,(USE hmin. INITIALLY, THEN USE d FROM PREVOIUS FOOTING,TO CALC. d).
Φ = 0.75

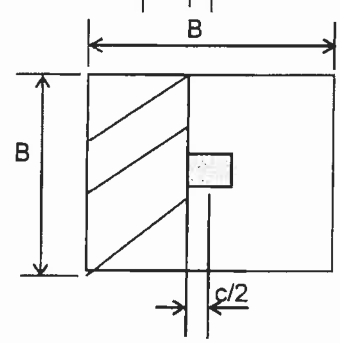
d_2 = Qu (A - (bo/12)^2) / (Φ*vc*4*bo)



ONE WAY SHEAR: ON CRITICAL SECTION d FROM FACE OF COLUMN

DETERMINE d REQUIRED FOR Pa ABOVE
vc = 2 x SQRT(f 'c)

Vu = Qu (B x(B/2 - d/12 -c/(2*12)))
ΦVc = Φ* vc*B*12*d   Φ = 0.75
Vu = Φ Vc
d_1 = Qu (B x(B/2 - d/12 -c/(2*12))) / (Φ*vc*B*12)
(USE hmin. INITIALLY, THEN USE d FROM PREVOIUS FOOTING,TO CALC. d).



AREA OF STEEL REQUIRED: CRITICAL SECTION AT FACE OF COLUMN

Mu = 1/2*Qu*B(B/2 - c/24)^2   Φ = 0.90
$Ru_{REQD.}$ =Mu / (Φ*B*d^2)   m = Fy/(.85*f 'c)
$As_{REQD.}$ =B*12*d*(1/m)*(1- SQRT(1 - 2*m*Ru / Fy))
ΦMn = Φ*As*Fy*(d - a/2)/12   a = As*Fy/(.85*f 'c*B*12)
Mu <= ΦMn

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SPREAD FOOTING DESIGN: PER ACI 318-02 AND 2003 IBC

f'c = 3000 PSI
Fy = 60,000 PSI
Qa = 2500 PSF
c = 5 IN.

LF = 1.5
CLR = 3 IN.
h_{MIN} = 12 IN.

SPREAD FOOTING BEARING CAPACITY & REINFORCEMENT

SIZE	B FT.	h IN.	Pa KIPS	REINFORCEMENT (NO. BARS) # 4	# 5	# 6	# 7	# 8	As_{REQD} IN.^{2}
2'-0" SQ.	2	12	10.0	3	2	2	1	1	0.518
3'-0" SQ.	3	12	22.5	4	3	2	2	1	0.778
4'-0" SQ.	4	12	40.0	6	4	3	2	2	1.037
5'-0" SQ.	5	14	62.5	8	5	4	3	2	1.512
6'-0" SQ.	6	17	90.0	12	8	5	4	3	2.203
7'-0" SQ.	7	18	122.5	14	9	7	5	4	2.722
8'-0" SQ.	8	22	160.0	20	13	9	7	5	3.802
9'-0" SQ.	9	23	202.5	23	15	11	8	6	4.471
10'-0" SQ.	10	26	250.0	29	19	13	10	8	5.616
11'-0" SQ.	11	28	302.5	34	22	16	12	9	6.653
12'-0" SQ.	12	31	360.0	41	27	19	14	11	8.035

Project:	Gila Country Sign Shop	Project Number:	30-09118-00
Subject:	MAXIUM HEIGHTS STEEL WALL FRAMING	Computed by:	EDN
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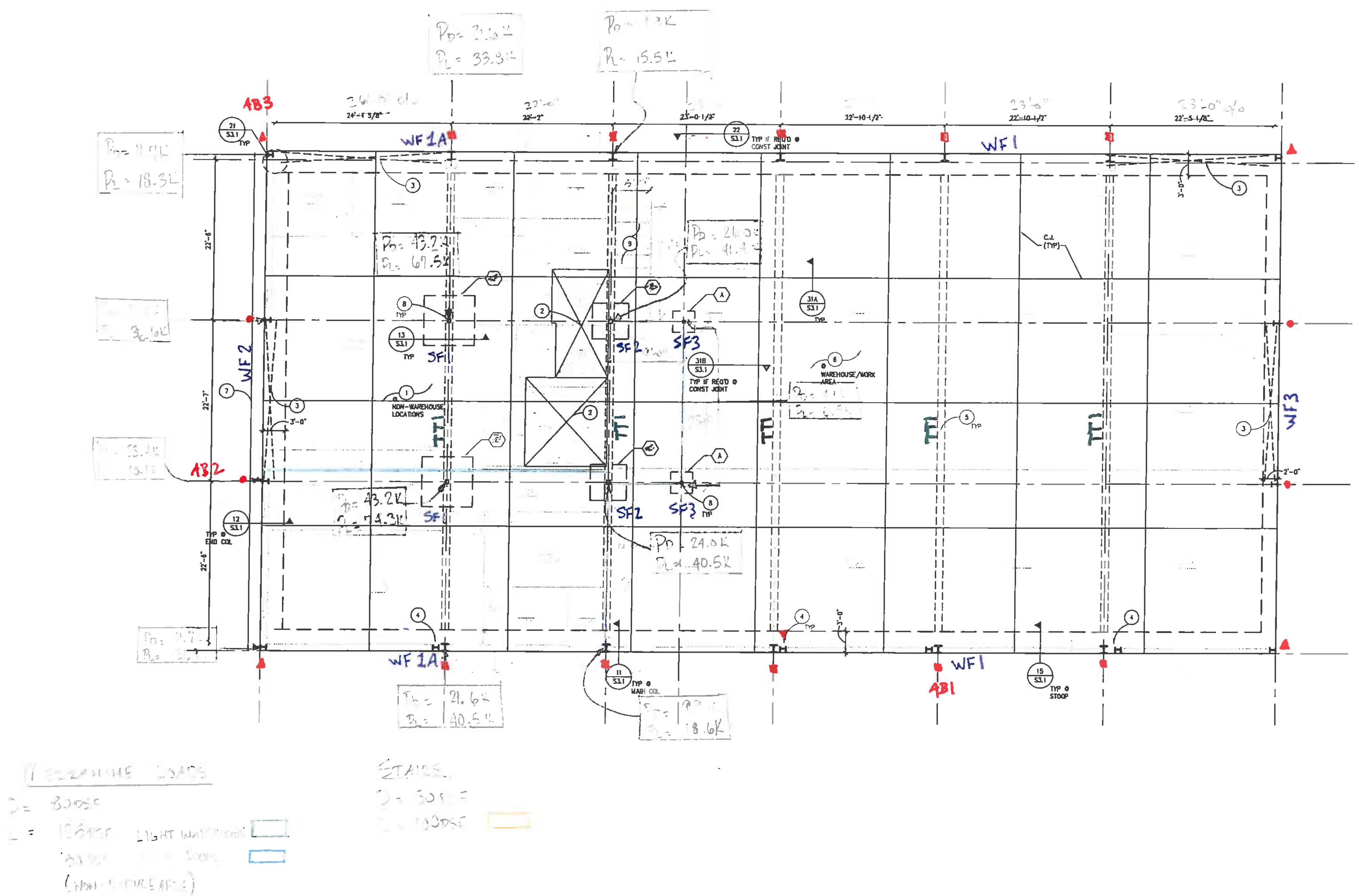
MAXIMUM HEIGHT OF INTERIOR NON-LOAD BEARING STEEL STUDS IN GYPBOARD WALLS, 5 PSF, L/240, NON-COMPOSITE, PER SSMA

STUD SIZE	MAXIMUM HEIGHT, FEET SPACING , INCHES 12	 16	 24	DESIGN BASED ON SSMA SIZE
Non-structural	17'-1"	15'-6"	13'-6"	362S125-33
	18'-5"	16'-9"	14'-7"	362S137-33
	21'-0"	19'-1"	16'-8"	362S162-43
Non-structural	18'-11"	17'-3"	15'-0"	400S125-33
	21'-8"	19'-8"	17'-2"	400S137-33
	22'-8"	20'-7"	18'-0"	400S162-43
Non-structural	26'-3"	23'-11"	20'-10"	600S125-33
	27'-5"	24'-11"	21'-9"	600S137-33
	31'-2"	28'-4"	24'-9"	600S162-43
Non-structural	36'-5"	33'-1"	28'-11"	800S125-43
	37'-10"	34'-4"	30'-0"	800S137-43
	41'-1"	37'-4"	32'-7"	800S200-43

STEEL STUD FRAMING INSTALLATION SHALL COMPLY WITH ASTM C 754

CONSULT SSMA PRODUCT TRECHNICAL INFORMATION FOR HEIGHT GREATER THAN SHOWN IN TABLE.

■ AB1 Group Moment Frame Columns (one edge)
● AB2 Group End Wall Columns (one edge)
▲ AB3 Group End Wall Columns (two edge)
▼ AB4 Group Portal Frame Columns (one edge 11 load)



FOUNDATION PLAN
SCALE: 1/8" = 1'-0"
NORTH

LEGEND NOTES

1. 4" CONCRETE SLAB-ON-GRADE ON 4" ABC. SEE ARCHITECTURAL FOR VAPOR RETARDER REQUIREMENTS FIN FL EL = 100'-0"
2. FLOOR SLAB SLOPED TO DRAIN LOCATION, FLOOR NOT DEPRESSED.
3. X-BRACED BAY BY PEBM MANUFACTURER
4. PORTAL BUILDING FRAME BY PEBM MANUFACTURER.
5. TENSION TIE, SEE DETAIL 32/S3.1
6. 5" CONCRETE SLAB-ON-GRADE WITH #4 @ 18" OC EA WAY CENTERED IN SLAB ON 4" ABC. SEE ARCHITECTURAL FOR VAPOR RETARDER REQUIREMENTS FIN FL EL = 100'-0"
7. PROVIDE 5-#6 LONGITUDINAL REBAR IN TOP AND BOTTOM OF GRADE BEAM FOOTING. SEE DETAIL 12/S3.3 FOR MORE INFORMATION.
8. ALL MEZZANINE FRAMING COMPONENTS BY PEBM MANUFACTURER.
9. STAIR FRAMING BY STAIR MANUFACTURER, COORDINATE LANDING SUPPORT WITH PEMB MANUFACTUER MEZZANINE FRAMING.

KEY PLAN

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FOUNDATION PLAN
GILA COUNTY FACILITIES & SIGN SHOP
CONSTRUCTION DOCUMENT SUBMITTAL

S1.1
30-09118-00
10/16/2009

DLR Group
Architecture Engineering Planning Interiors

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Created : Tue Oct 20 20:00:57 2009, System Optima 1.3.2

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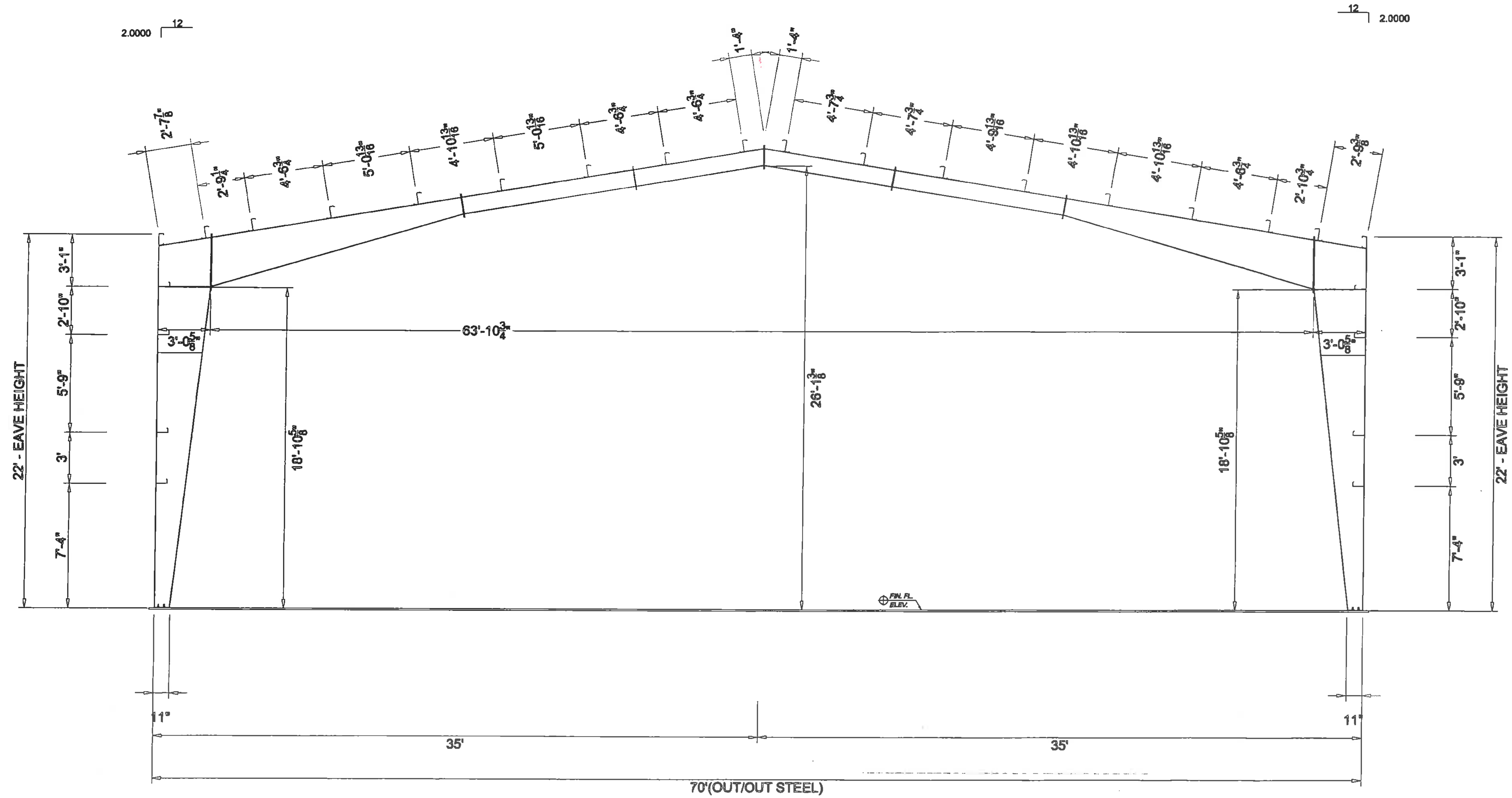

This drawing is not for construction. This drawing is Intended to depict general building information and is solely for sales presentation purposes. For clarity of presentation, items depicted may be different from actual design and final drawings. In the event of conflict between this drawing and the purchase order, the purchase order shall prevail.

CROSS SECTION AT FRAME LINE "3" - (A) SIGN SHOP

C B D A

DESIGNS SHOWN ARE BASED ON THE BASIC BUILDING ITSELF, THEY DO NOT INCLUDE LOADS FROM ANY BUILDING OPTIONS OR ANY OTHER MATERIAL

FRAME CLEARANCES SHOWN ARE APPROXIMATE AND MAY VARY DUE TO FIELD CONDITIONS AND LOADS.
VERTICAL CLEARANCE DIMENSIONS ARE FROM FINISHED FLOOR REFERENCE ELEVATION.

DESIGN DATA/ANCHOR ROD DESIGN BASED ON

MANUFACTURER RESERVES THE RIGHT TO CHANGE THE FINAL DESIGN. IF DESIGN INFORMATION (IE. CLEARANCES, BASE PLATE/ANCHOR ROD DESIGN) IS TO BE USED FOR CONSTRUCTION MANUFACTURER MUST BE NOTIFIED PRIOR TO ACCEPTANCE OF ORDER.

CONTACT SALES ENGINEER FOR REVIEW BEFORE USING THIS INFORMATION FOR CONSTRUCTION

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ENGINEERING CERTIFICATION OF MATERIALS SUPPLIED BY MANUFACTURER WILL BE PROVIDED BY SEAL AND SIGNATURE OF LICENSED ENGINEER ON FINAL ERECTION DRAWINGS.

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Wynn	
Best Fit	10/20/09

METALLIC BUILDING COMPANY

Architectural Building Systems, LLC	Gila County
1720 W. Lincoln Street	
Phoenix, AZ 85007	
COUNTY:	Globe, AZ
CONTACT: Wynn Pratt	

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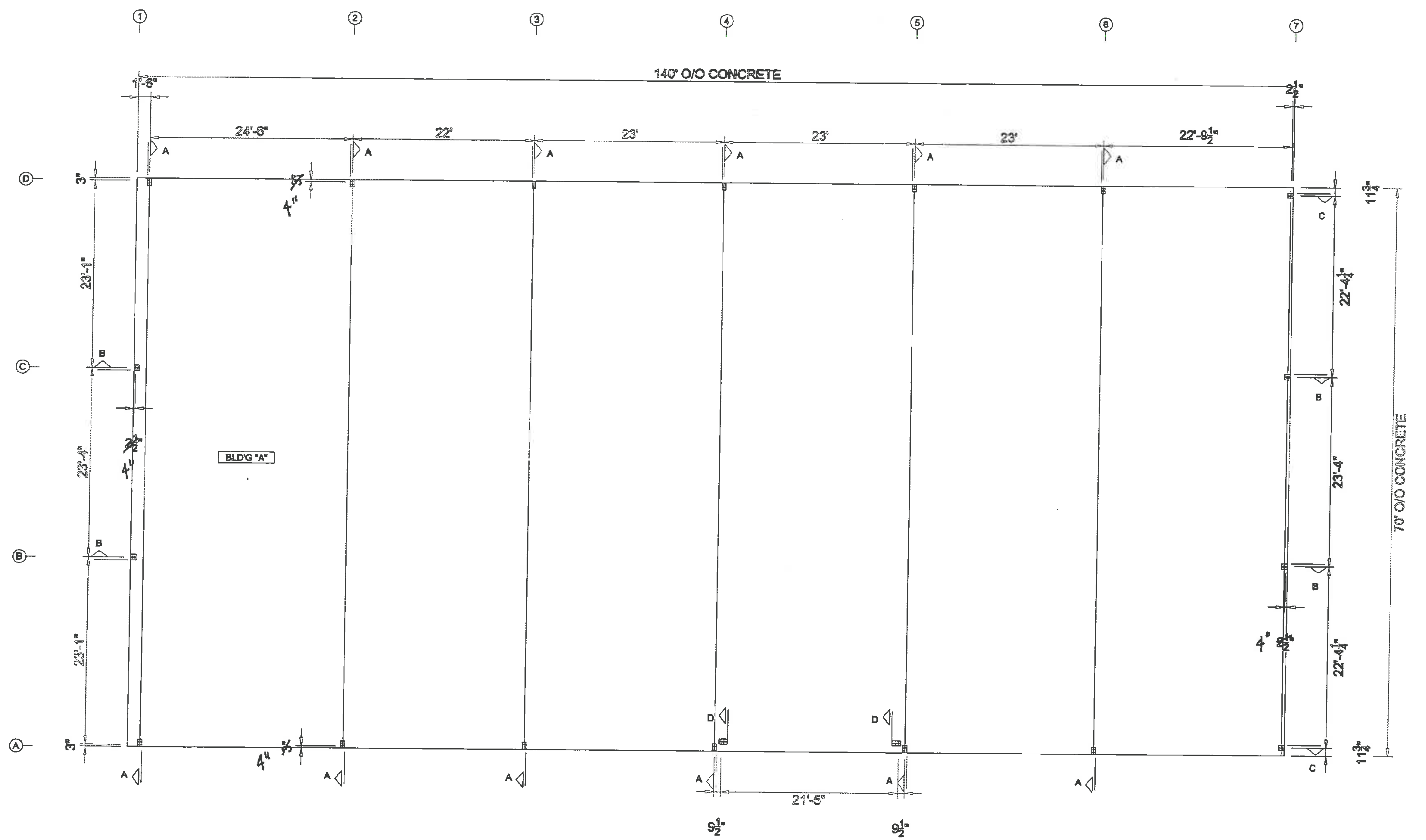
NOT FOR CONSTRUCTION

ACCESSORY SCHEDULE

Mark	Description	Qty.



This drawing is not for construction. This drawing is intended to depict general building information and is solely for sales presentation purposes. For clarity of presentation, items depicted may be different from actual design and final drawings. In the event of conflict between this drawing and the purchase order, the purchase order shall prevail.

ANCHOR ROD PLAN - (A) SIGN SHOP

C, B, D, A

DESIGNS SHOWN ARE BASED ON THE BASIC BUILDING ITSELF, THEY DO NOT INCLUDE LOADS FROM ANY BUILDING OPTIONS OR ANY OTHER MATERIAL

FOUNDATION MUST BE SQUARE AND LEVEL. ALL ANCHOR RODS MUST BE TRUE IN SIZE, LOCATION, AND PROJECTION. ANCHOR ROD PROJECTIONS MUST BE HELD TO KEEP THREADS CLEAR OF FINISHED CONCRETE.

ENGINEERING CERTIFICATION OF MATERIALS SUPPLIED BY MANUFACTURER WILL BE PROVIDED BY SEAL AND SIGNATURE OF LICENSED ENGINEER ON FINAL ERECTION DRAWINGS.

THIS DRAWING IS FOR ANCHOR ROD PLACEMENT ONLY AND IS NOT A SUBSTITUTE FOR FOUNDATION DESIGN.

CONTACT SALES ENGINEER FOR REVIEW BEFORE USING THIS INFORMATION FOR CONSTRUCTION

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	22x34
Wynn	
Best Fit	10/20/09

METALLIC BUILDING COMPANY

Architectural Building Systems, LLC
1720 W. Lincoln Street
Phoenix, AZ 85007
COUNTY:
CONTACT: Wynn Pratt

Gila County
Globe, AZ

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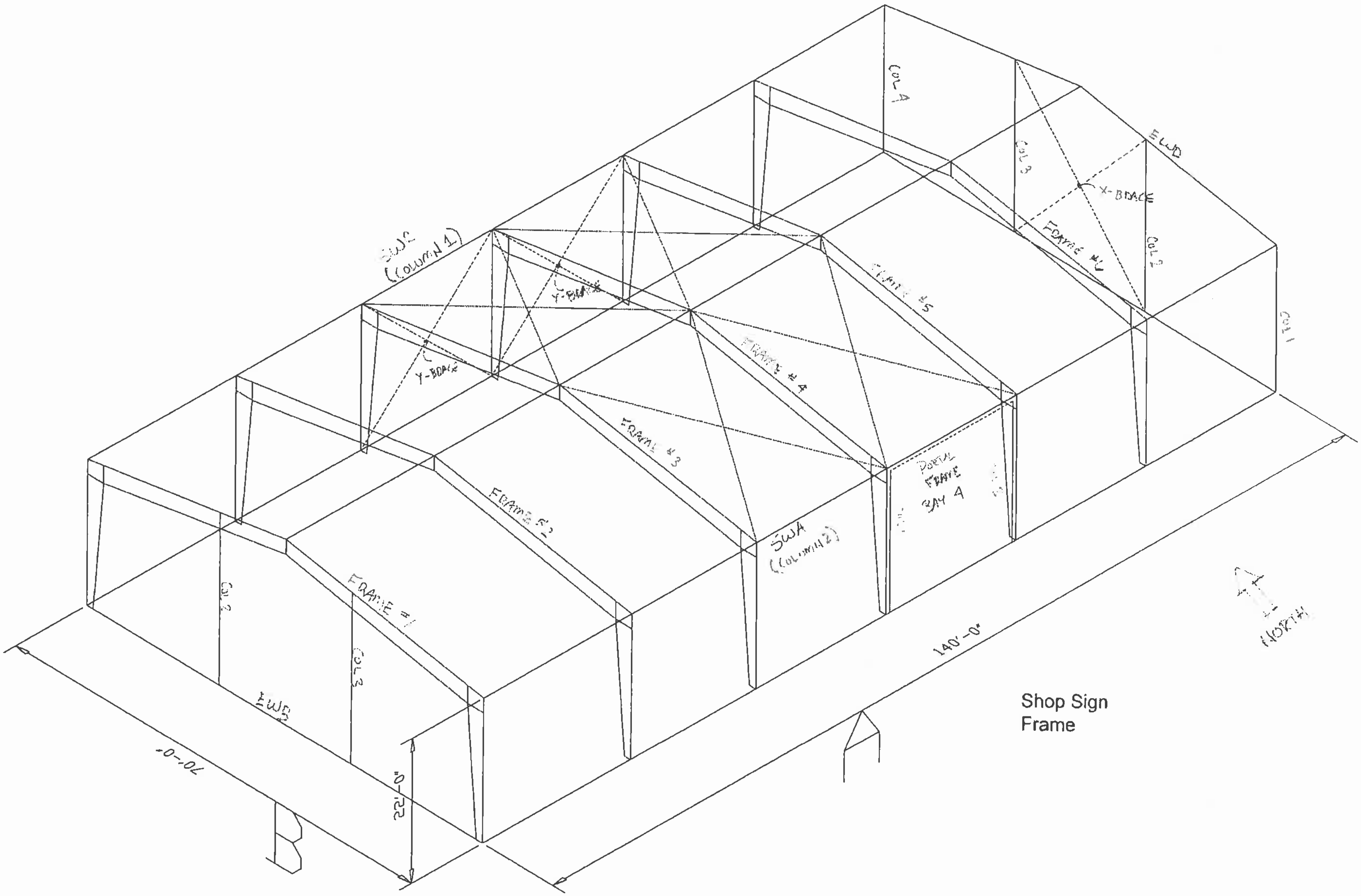

Shop Sign
Frame

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Design Report Optima 1.3.2 38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

Architectural Building Systems, LLC

Note: All design information provided is preliminary, including but not limited to "Designed", "System Standard" and "Default" design criteria. The Manufacturer will not be responsible for conditions resulting from changes in the final design unless that specific requirement is noted on the Purchase Order.

BUILDING CODE

Project Use Category:	Commercial	**Jobsite State:**	AZ
Building Code:	2006 IBC	**Jobsite County:**	Gila
		Jobsite City:	Globe

Live/Wind

Live Load:	20.000 psf		
Reduction:	No		
Wind Load:	90.00 mph	**Wind Category:**	N/A
Wind Exposure:	Exposure C	**Miles From Coastline:**	N/A
Hurricane Coastline:	No	**Rain Intensity:**	5.0000 in/hr

Snow

Ground Snow Load:	0.000 psf	**Snow Exposure:**	N/A
Min Roof Snow Load:	0.000 psf	**Rain Load:**	N/A
Thermal Condition:	N/A	**Sea Level Elevation:**	N/A

Seismic

Spectral Response(Ss):	37.40 %	**% of Snow Load for Seismic:**	Normal
Spectral Response(Sh):	N/A	**Seismic Zone:**	N/A
Spectral Response(S1):	10.20 %	**Near Source Factor:**	N/A
Spectral Response(S2):	N/A	**Design Seismic For Schools:**	N/A
Velocity Coefficient(Aa):	N/A	**Site Class/Soil Type:**	(D) Stiff Soil
Accelerated Coefficient(Av):	N/A		

NOTE:

MEZZANINE LOADS ARE NOT INCORPORATED INTO PRELIMINARY CALCULATIONS THEREFORE; ASSUMED MEZZANINE LOADS ARE ESTABLISHED BY FOUNDATION ENGINEER AND THESE LOADS ARE NOTED ON THE PLAN SHEETS AND INCLUDED IN THE FOUNDATION CALCULATION DESIGN

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METALLIC BUILDING COMPANY

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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

BUILDING A - SIGN SHOP

Label:	A	**Type:**	Stand Alone
Structure:	New	**Frame Type:**	Symmetrical
		Elevation A:	Sidewall

GEOMETRY, SIDEWALLS & ENDWALLS

Width: 70'-0"

Length: 140'-0"

SWA

Eave Height:	22'-0"
Roof Slope:	2.000000 / 12
Dist. to Ridge:	35'-0"
Girts:	8.0" - Flush

SWC

Eave Height:	22'-0"
Roof Slope:	2.000000 / 12
Dist. to Ridge:	35'-0"
Girts:	8.0" - Flush

EWB

Type:	Non-Expandable Frame
Girts:	8.0" - Flush
Setback:	System Standard

EWD

Type:	Bearing Frame with Hot-Rolled Raft
Girts:	8.0" - Flush
Setback:	System Standard

Purlins:	8.0" Z	**Pregalv. Secondary:**	No
mary Steel Shop Coat:	Red	**Frame Bolt Washers:**	No

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ɪUILDING A - SIGN SHOP

ɪPACINGS:

Bay Spacing (EWB-EWD): 26'-0", 22'-0", 4@23'-0"
EWB COL. Spacing (SWC-SWA): 3@23'-4"
EWD COL. Spacing (SWA-SWC): 3@23'-4"

SWA Girt Locations (Base to Eave): System Standard:	7'-4",	10'-4",	16'-1",	18'-11"	
SWC Girt Locations (Base to Eave): System Standard:	7'-4",	10'-4",	16'-1",	18'-11"	
EWB Girt Locations (Base to Peak): System Standard:	7'-4",	10'-4",	16'-1",	20'-11",	26'-2 1/4"
EWD Girt Locations (Base to Peak): System Standard:	7'-4",	10'-4",	16'-1",	20'-11",	26'-2 1/4"

Purlin Spacing: System Standard
Purlin Locations (SWA - Eave to Peak): 2'-10 3/4", 5'-9 1/2", 10'-4 1/4", 15'-3 1/16", 20'-1 7/8", 24'-11 5/8", 29'-7 7/16", 34'-3 3/16"

Purlin Locations (SWC - Eave to Peak): 2'-9 1/4", 5'-6 1/2", 10'-1 1/4", 15'-2 1/16", 20'-0 7/8", 25'-1 11/16", 29'-8 7/16", 34'-3 3/16"

**Note: Purlin and girt depths and locations are supplied for reference only, and may be changed at Manufacturer's discretion without ce unless specifically stated otherwise in the "Notes" section of this document.

꞉RAME GROUPS

Group Number: 1

Frame Lines: 1, 2, 3, 4, 5, 6

SWA

Column: Tapered Allowed
Max Col. Web Depth: 68.00"
Max Raf. Web Depth: 68.00"
Ext Col. Elevation: At Finished Floor

SWC

Column: Tapered Allowed
Max Col. Web Depth: 68.00"
Max Raf. Web Depth: 68.00"
Ext Col. Elevation: At Finished Floor

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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

ꞁuilding A - SIGN SHOP

.OADS, WIND ENCLOSURE, DEFLECTIONS & DRIFTS

Building Loads

Roof Snow Load By Design:	0.000 psf
Occupancy Category:	II - Normal
Thermal Condition:	N/A
Seismic Design Category:	C

Importance Factors

Snow Is:	1.00
Wind Iw:	1.00
Seismic Ie:	1.00
Designed Snow Exposure:	

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Wind Enclosure

Wind Enclosure:	Calculated - Enclosed
Are all Framed Openings enclosed with materials designed to resist building wind loads:	Yes
Are all Open Areas for Other enclosed with materials designed to resist building wind loads:	Yes

Uniform Collateral Loads

Ceiling Load:	0.000 psf	Other:	5.000 psf
Plaster/Sheetrock Ceiling:	No		
Brittle/Dryvit:	No		

Deflections

Purlins Live:	L/150 - Default	Rafters Live:	L/150 - Default
Purlins Snow:	L/180 - Default	Rafters Snow:	L/180 - Default
lins Wind:	L/180 - Default	Rafters Wind:	L/180 - Default
.lins Total Gravity:	L/120 - Default	Rafters Total Gravity:	L/120 - Default
Purlins Total Uplift:	L/N/A - Default	Rafters Total Uplift:	L/N/A - Default
Girts:	L/120 - Default	Endwall Columns:	L/120 - Default

Drifts

Portal Frame Wind:	H/60 - Default
Portal Frame Seismic:	H/50 - Default
Crane:	H/100 - Default
Frame Live:	H/60 - Default
Frame Snow:	H/60 - Default
Frame Wind:	H/60 - Default
Frame Seismic:	H/50 - Default
Frame Total Gravity:	H/60 - Default
Frame Total Wind:	H/60 - Default
Frame Total Seismic:	H/50 - Default

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ɜUILDING A - SIGN SHOP

ɜRACING

SWA	Portal Frame	**(EWB to EWD) @ Bays:**	4
Roof:	Cable	**(EWB to EWD) @ Bays:**	3, 4
SWC:	Cable	**(EWD to EWB) @ Bays:**	4, 3
EWB:	None	**(SWC to SWA) @ Bays:**	No Bays
EWD:	Cable	**(SWA to SWC) @ Bays:**	2
Purlin:	Angles		
Girt:	None		

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ꓤOOF PANEL (9,936 sqft)

Type:	Ultra-Dek
Gauge:	24
Thickness:	N/A
Color:	SIG - 300 TBD*
Finish Warranty:	Yes
Interior Panel:	N/A

Fastener Information

Type:	Self-Drilling
Head Finish:	Long-Life
Length:	Standard

Options

SS Clip Type:	N/A
Thermal Blocks:	N/A
UL90:	N/A
UL Letter:	N/A
IBL Tools:	No
	N/A
	No
	N/A

VALL PANEL (9,445 sqft)

e:	PBR
Gauge:	26
Color:	SIG - 200 TBD
Thickness:	N/A
Finish Warranty:	N/A
Interior Panel:	

Fastener Information

Type:	Self-Drilling
Head Finish:	Standard
Length:	1-1/4"

Options

Concrete Notch:	No

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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

UILDING A - SIGN SHOP

ESIGN DATA FRAME(S): PORTAL - BAY 4

Inside Clearance:	19'-10"	**Peak Clearance:**	19'-3 15/16"
		Peak Rafter Depth:	18.00"

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Column 1 (Portal) Elevation: A

Column Depth		**Knee**	
Base:	12.06"	Rafter Depth:	18.00"
Knee:	12.06"	Clearance:	19'-4"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	12.50"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"

Maximum Reactions		
Vertical: WIND ONLY	6.74 Kips	-5.64 Kips
Horizontal: WIND ONLY	2.98 Kips (LONG +/- x) TO FTG	-3.10 Kips
Longitudinal:	0.00 Kips (HORIZ +/- y) TO FTG	0.00 Kips

Column 2 (Portal) Elevation: A

Column Depth		**Knee**	
ase:	12.06"	Rafter Depth:	18.00"
nee:	12.06"	Clearance:	19'-4"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	12.50"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"

Maximum Reactions		
Vertical:	6.74 Kips	-5.64 Kips
Horizontal:	3.10 Kips	-2.98 Kips
Longitudinal:	0.00 Kips	0.00 Kips

```
*************************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which  *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                          *
*************************************************************************************
```

ESIGN DATA FRAME(S): 1

Inside Clearance:	64'-9 1/2"	**Peak Clearance:**	26'-1 3/8"
		Peak Rafter Depth:	12.00"

Column 1 (SWC)

Column Depth		**Knee**	
Base:	10.56"	Rafter Depth:	26.56"
Knee:	30.56"	Clearance:	19'-5 3/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	6.00"
Gauge:	4.00"	Thickness:	0.38"

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BUILDING A - SIGN SHOP

Column 1 (Sidewall)

Maximum Reactions		
Vertical:	14.42 Kips	-8.15 Kips
Horizontal:	7.87 Kips	-5.92 Kips
Longitudinal:	0.00 Kips	0.00 Kips

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Column 2 (SWA)

Column Depth		Knee	
Base:	10.56"	Rafter Depth:	26.56"
Knee:	30.56"	Clearance:	19'-5 3/4"

Anchor Rods		Base Plate:	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	6.00"
Gauge:	4.00"	Thickness:	0.38"

Maximum Reactions		
Vertical:	14.42 Kips	-8.15 Kips
Horizontal:	5.92 Kips	-7.87 Kips
Longitudinal:	0.00 Kips	0.00 Kips

```
******************************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which    *
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  produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.             *
* Anchor rods are not supplied by Manufacturer.                                          *
******************************************************************************************
```
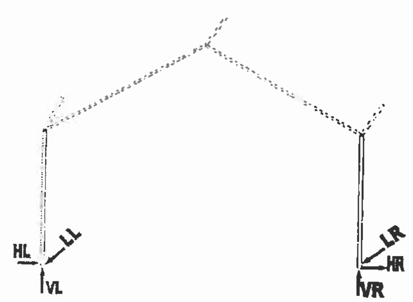

Column 1 (SWC) -->



ndividual Loads - Unfactored

C-lumn 1 (SWC)	Vertical	Horizontal	Longitudinal
ral Primary Wind Load 1	-9.531 Kips	-6.558 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-6.712 Kips	-5.987 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-6.207 Kips	-1.507 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-3.388 Kips	-0.936 Kips	-0.000 Kips
Lateral Seismic Load	-0.395 Kips	-0.671 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-7.750 Kips	-2.042 Kips	-0.000 Kips

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BUILDING A - SIGN SHOP

Individual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Longitudinal Primary Wind Load 2	-6.961 Kips	-2.223 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-4.701 Kips	-0.669 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-4.369 Kips	-0.745 Kips	-0.000 Kips
Roof Collateral Load	2.406 Kips	1.333 Kips	-0.000 Kips
Roof Dead Load	2.294 Kips	1.055 Kips	-0.000 Kips
Roof Live Load	9.625 Kips	5.332 Kips	-0.000 Kips
Column 2 (SWA)			
Lateral Primary Wind Load 1	-6.207 Kips	1.507 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-3.388 Kips	0.936 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-9.531 Kips	6.558 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-6.712 Kips	5.987 Kips	-0.000 Kips
Lateral Seismic Load	0.395 Kips	-0.671 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-6.961 Kips	2.223 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-7.750 Kips	2.042 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-4.369 Kips	0.745 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-4.701 Kips	0.669 Kips	-0.000 Kips
Roof Collateral Load	2.406 Kips	-1.333 Kips	-0.000 Kips
Roof Dead Load	2.294 Kips	-1.055 Kips	-0.000 Kips
Roof Live Load	9.625 Kips	-5.332 Kips	-0.000 Kips

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METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

)ESIGN DATA FRAME(S): 2

Inside Clearance:	63'-9 1/2"	**Peak Clearance:**	26'-1 3/8"
		Peak Rafter Depth:	12.00"

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Column 1 (SWC)

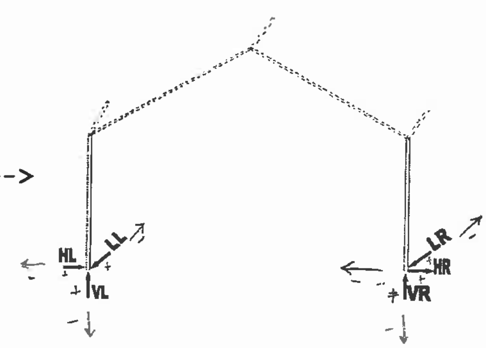
Column Depth		**Knee**	
Base:	10.63"	Rafter Depth:	35.56"
Knee:	36.63"	Clearance:	18'-9 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	6.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	23.86 Kips	-11.08 Kips	
Horizontal:	13.73 Kips	-6.74 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
ase:	10.63"	Rafter Depth:	35.56"
.nee:	36.63"	Clearance:	18'-9 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	6.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	23.86 Kips	-11.08 Kips	
Horizontal:	6.74 Kips	-13.73 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

```
*******************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which    *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.             *
* Anchor rods are not supplied by Manufacturer.                                          *
*******************************************************************************
```

Column 1 (SWC) -->



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ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Lateral Primary Wind Load 1	-11.395 Kips	-7.746 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-6.620 Kips	-6.686 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-7.838 Kips	-2.000 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-3.063 Kips	-0.940 Kips	-0.000 Kips
Lateral Seismic Load	-0.616 Kips	-1.064 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-13.098 Kips	-3.687 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.781 Kips	-3.987 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.951 Kips	-1.267 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.397 Kips	-1.393 Kips	-0.000 Kips
Roof Collateral Load	4.069 Kips	2.356 Kips	-0.000 Kips
Roof Dead Load	3.362 Kips	1.678 Kips	-0.000 Kips
Roof Live Load	16.275 Kips	9.425 Kips	-0.000 Kips
Column 2 (SWA)			
Lateral Primary Wind Load 1	-7.838 Kips	2.000 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-3.063 Kips	0.940 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-11.395 Kips	7.746 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-6.620 Kips	6.686 Kips	-0.000 Kips
Lateral Seismic Load	0.616 Kips	-1.065 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.781 Kips	3.987 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-13.098 Kips	3.687 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.397 Kips	1.393 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.951 Kips	1.267 Kips	-0.000 Kips
Roof Collateral Load	4.069 Kips	-2.356 Kips	-0.000 Kips
R~of Dead Load	3.362 Kips	-1.678 Kips	-0.000 Kips
f Live Load	16.275 Kips	-9.425 Kips	-0.000 Kips

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DESIGN DATA FRAME(S): 3

Inside Clearance:	63'-9 1/2"	**Peak Clearance:**	26'-1 3/8"
		Peak Rafter Depth:	12.00"

Column 1 (SWC)

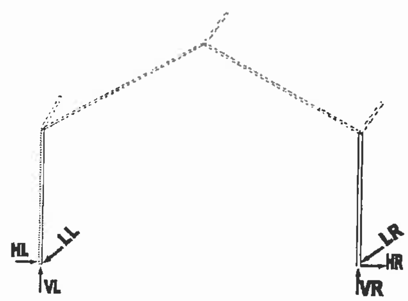
Column Depth		**Knee**	
Base:	10.63"	Rafter Depth:	34.56"
Knee:	36.63"	Clearance:	18'-10 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	6.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	23.12 Kips	-13.48 Kips	
Horizontal:	13.27 Kips	-6.51 Kips	
Longitudinal:	0.00 Kips	-2.90 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
Base:	10.63"	Rafter Depth:	34.56"
Knee:	36.63"	Clearance:	18'-10 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	6.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	23.12 Kips	-10.69 Kips	
Horizontal:	6.51 Kips	-13.27 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

```
*****************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which   *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                          *
*****************************************************************************
```

Column 1 (SWC) -->



ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Brace Downward forces due to Longitudinal Wind	2.781 Kips	-0.012 Kips	-0.000 Kips
Brace Downward forces due to Seismic	2.870 Kips	-0.012 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	-2.781 Kips	0.014 Kips	-2.897 Kips
Brace Upward forces due to Seismic	-2.870 Kips	0.015 Kips	-2.990 Kips
Lateral Primary Wind Load 1	-11.025 Kips	-7.488 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-6.405 Kips	-6.466 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-7.584 Kips	-1.928 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-2.963 Kips	-0.906 Kips	-0.000 Kips
Lateral Seismic Load	-0.600 Kips	-1.035 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-12.672 Kips	-3.560 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.398 Kips	-3.850 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.693 Kips	-1.221 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.157 Kips	-1.343 Kips	-0.000 Kips
Roof Collateral Load	3.937 Kips	2.277 Kips	-0.000 Kips
Roof Dead Load	3.289 Kips	1.632 Kips	-0.000 Kips
Roof Live Load	15.750 Kips	9.106 Kips	-0.000 Kips
Column 2 (SWA)			
Brace Downward forces due to Longitudinal Wind	-0.010 Kips	0.012 Kips	-0.000 Kips
Brace Downward forces due to Seismic	-0.010 Kips	0.012 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	0.010 Kips	-0.014 Kips	-0.000 Kips
Brace Upward forces due to Seismic	0.010 Kips	-0.015 Kips	-0.000 Kips
Lateral Primary Wind Load 1	-7.584 Kips	1.928 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-2.963 Kips	0.906 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-11.025 Kips	7.488 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-6.405 Kips	6.466 Kips	-0.000 Kips
Lateral Seismic Load	0.600 Kips	-1.035 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.398 Kips	3.850 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-12.672 Kips	3.560 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.157 Kips	1.343 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.693 Kips	1.221 Kips	-0.000 Kips
Roof Collateral Load	3.938 Kips	-2.277 Kips	-0.000 Kips
Roof Dead Load	3.289 Kips	-1.632 Kips	-0.000 Kips
Roof Live Load	15.750 Kips	-9.106 Kips	-0.000 Kips

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METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

ESIGN DATA FRAME(S): 4

Inside Clearance:	63'-9 1/2"	**Peak Clearance:**	26'-1 3/8"
		Peak Rafter Depth:	12.00"

Column 1 (SWC)

Column Depth		**Knee**	
Base:	10.63"	**Rafter Depth:**	35.56"
Knee:	36.63"	**Clearance:**	18'-9 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	**Length:**	11.00"
Diameter:	0.75"	**Width:**	6.00"
Gauge:	4.00"	**Thickness:**	0.38"

Maximum Reactions		
Vertical:	23.61 Kips	-13.73 Kips
Horizontal:	13.58 Kips	-6.66 Kips
Longitudinal:	0.00 Kips	-2.90 Kips

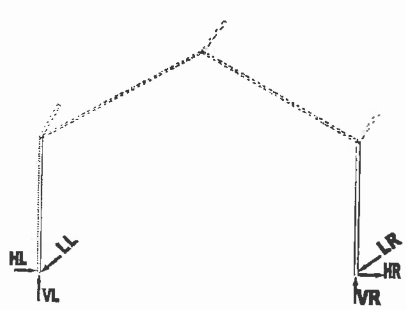
Column 2 (SWA)

Column Depth		**Knee**	
ase:	10.63"	**Rafter Depth:**	35.56"
nee:	36.63"	**Clearance:**	18'-9 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	**Length:**	11.00"
Diameter:	0.75"	**Width:**	6.00"
Gauge:	4.00"	**Thickness:**	0.38"

Maximum Reactions		
Vertical:	23.61 Kips	-10.94 Kips
Horizontal:	6.66 Kips	-13.58 Kips
Longitudinal:	0.00 Kips	0.00 Kips

```
*****************************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which   *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                         *
*****************************************************************************************
```

Column 1 (SWC) -->



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Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Brace Downward forces due to Longitudinal Wind	2.781 Kips	-0.012 Kips	-0.000 Kips
Brace Downward forces due to Seismic	2.870 Kips	-0.012 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	-2.781 Kips	0.014 Kips	-2.897 Kips
Brace Upward forces due to Seismic	-2.870 Kips	0.015 Kips	-2.990 Kips
Lateral Primary Wind Load 1	-11.273 Kips	-7.663 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-6.549 Kips	-6.614 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-7.754 Kips	-1.979 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-3.030 Kips	-0.930 Kips	-0.000 Kips
Lateral Seismic Load	-0.610 Kips	-1.055 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-12.957 Kips	-3.648 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.654 Kips	-3.944 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.866 Kips	-1.253 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.317 Kips	-1.378 Kips	-0.000 Kips
Roof Collateral Load	4.025 Kips	2.331 Kips	-0.000 Kips
Roof Dead Load	3.339 Kips	1.664 Kips	-0.000 Kips
Roof Live Load	16.100 Kips	9.324 Kips	-0.000 Kips
Column 2 (SWA)			
Brace Downward forces due to Longitudinal Wind	-0.010 Kips	0.012 Kips	-0.000 Kips
Brace Downward forces due to Seismic	-0.010 Kips	0.012 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	0.010 Kips	-0.014 Kips	-0.000 Kips
Brace Upward forces due to Seismic	0.010 Kips	-0.015 Kips	-0.000 Kips
Lateral Primary Wind Load 1	-7.754 Kips	1.979 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-3.030 Kips	0.930 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-11.273 Kips	7.663 Kips	-0.000 Kips
ral Primary Wind Load 4	-6.549 Kips	6.614 Kips	-0.000 Kips
eral Seismic Load	0.610 Kips	-1.055 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.654 Kips	3.944 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-12.957 Kips	3.648 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.317 Kips	1.378 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.866 Kips	1.253 Kips	-0.000 Kips
Roof Collateral Load	4.025 Kips	-2.331 Kips	-0.000 Kips
Roof Dead Load	3.339 Kips	-1.664 Kips	-0.000 Kips
Roof Live Load	16.100 Kips	-9.324 Kips	-0.000 Kips

METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

)ESIGN DATA FRAME(S): 5

Inside Clearance:	63'-9 1/2"	**Peak Clearance:**	26'-1 3/8"
		Peak Rafter Depth:	12.00"

Column 1 (SWC)

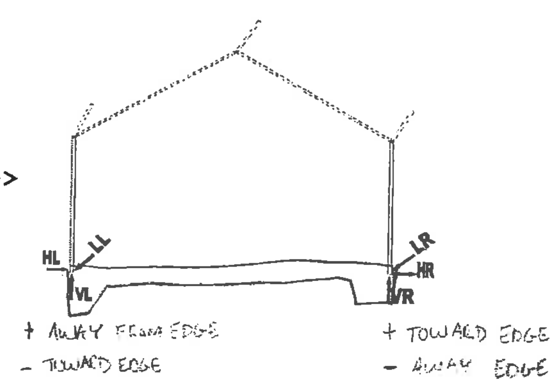
Column Depth		**Knee**	
Base:	10.63"	**Rafter Depth:**	35.56"
Knee:	36.63"	**Clearance:**	18'-9 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	**Length:**	11.00"
Diameter:	0.75"	**Width:**	6.00"
Gauge:	4.00"	**Thickness:**	0.38"
Maximum Reactions			
Vertical:	23.61 Kips	-13.73 Kips	
Horizontal:	13.58 Kips	-6.66 Kips	
Longitudinal:	0.00 Kips	-2.90 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
'ase:	10.63"	**Rafter Depth:**	35.56"
.nee:	36.63"	**Clearance:**	18'-9 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	**Length:**	11.00"
Diameter:	0.75"	**Width:**	6.00"
Gauge:	4.00"	**Thickness:**	0.38"
Maximum Reactions			
Vertical:	23.61 Kips	-10.94 Kips	
Horizontal:	6.66 Kips	-13.58 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

```
***************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which  *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                          *
***************************************************************************
```

Column 1 (SWC) -->



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ndividual Loads - Unfactored

ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Brace Downward forces due to Longitudinal Wind	2.781 Kips	-0.012 Kips	-0.000 Kips
Brace Downward forces due to Seismic	2.870 Kips	-0.012 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	-2.781 Kips	0.014 Kips	-2.897 Kips
Brace Upward forces due to Seismic	-2.870 Kips	0.015 Kips	-2.990 Kips
Lateral Primary Wind Load 1	-11.273 Kips	-7.663 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-6.549 Kips	-6.614 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-7.754 Kips	-1.979 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-3.030 Kips	-0.930 Kips	-0.000 Kips
Lateral Seismic Load	-0.610 Kips	-1.055 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-12.957 Kips	-3.648 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.654 Kips	-3.944 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.866 Kips	-1.253 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.317 Kips	-1.378 Kips	-0.000 Kips
Roof Collateral Load	4.025 Kips	2.331 Kips	-0.000 Kips
Roof Dead Load	3.339 Kips	1.664 Kips	-0.000 Kips
Roof Live Load	16.100 Kips	9.324 Kips	-0.000 Kips
Column 2 (SWA)			
Brace Downward forces due to Longitudinal Wind	-0.010 Kips	0.012 Kips	-0.000 Kips
Brace Downward forces due to Seismic	-0.010 Kips	0.012 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	0.010 Kips	-0.014 Kips	-0.000 Kips
Brace Upward forces due to Seismic	0.010 Kips	-0.015 Kips	-0.000 Kips
Lateral Primary Wind Load 1	-7.754 Kips	1.979 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-3.030 Kips	0.930 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-11.273 Kips	7.663 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-6.549 Kips	6.614 Kips	-0.000 Kips
Lateral Seismic Load	0.610 Kips	-1.055 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.654 Kips	3.944 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-12.957 Kips	3.648 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.317 Kips	1.378 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.866 Kips	1.253 Kips	-0.000 Kips
Roof Collateral Load	4.025 Kips	-2.331 Kips	-0.000 Kips
Roof Dead Load	3.339 Kips	-1.664 Kips	-0.000 Kips
Roof Live Load	16.100 Kips	-9.324 Kips	-0.000 Kips

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Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

)ESIGN DATA FRAME(S): 6

Inside Clearance:	63'-9 1/2"	**Peak Clearance:**	26'-1 3/8"
		Peak Rafter Depth:	12.00"

Column 1 (SWC)

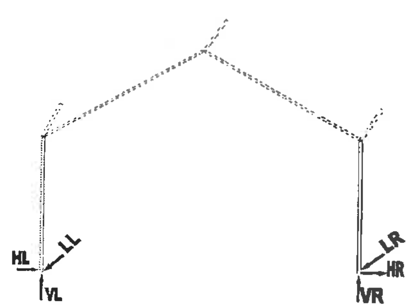
Column Depth		**Knee**	
Base:	10.63"	Rafter Depth:	35.56"
Knee:	36.63"	Clearance:	18'-9 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	6.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	23.45 Kips	-10.87 Kips	
Horizontal:	13.48 Kips	-6.61 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
'ase:	10.63"	Rafter Depth:	35.56"
.nee:	36.63"	Clearance:	18'-9 5/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	6.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	23.45 Kips	-10.87 Kips	
Horizontal:	6.61 Kips	-13.48 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

```
*****************************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which   *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.             *
* Anchor rods are not supplied by Manufacturer.                                           *
*****************************************************************************************
```

Column 1 (SWC) -->



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ndividual Loads - Unfactored

ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Lateral Primary Wind Load 1	-11.191 Kips	-7.607 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-6.501 Kips	-6.566 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-7.697 Kips	-1.964 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-3.008 Kips	-0.924 Kips	-0.000 Kips
Lateral Seismic Load	-0.606 Kips	-1.048 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-12.863 Kips	-3.621 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.569 Kips	-3.915 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.809 Kips	-1.244 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.264 Kips	-1.368 Kips	-0.000 Kips
Roof Collateral Load	3.996 Kips	2.314 Kips	-0.000 Kips
Roof Dead Load	3.324 Kips	1.656 Kips	-0.000 Kips
Roof Live Load	15.983 Kips	9.256 Kips	-0.000 Kips
Column 2 (SWA)			
Lateral Primary Wind Load 1	-7.697 Kips	1.964 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-3.008 Kips	0.924 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-11.191 Kips	7.607 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-6.501 Kips	6.566 Kips	-0.000 Kips
Lateral Seismic Load	0.606 Kips	-1.048 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.569 Kips	3.916 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-12.863 Kips	3.621 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.264 Kips	1.368 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.809 Kips	1.244 Kips	-0.000 Kips
Roof Collateral Load	3.996 Kips	-2.314 Kips	-0.000 Kips
R‑oof Dead Load	3.324 Kips	-1.656 Kips	-0.000 Kips
f Live Load	15.983 Kips	-9.256 Kips	-0.000 Kips

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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

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ESIGN DATA ENDWALL(s): EWB

Column 2 (Hot Rolled)

Anchor Rods:	4	Base Plate Width:	8.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.13"
Column Depth:	8.11"	Base Plate Thickness:	0.38"
Flange Width:	4.02"		

Column 3 (Hot Rolled)

Anchor Rods:	4	Base Plate Width:	8.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.13"
Column Depth:	8.11"	Base Plate Thickness:	0.38"
Flange Width:	4.02"		

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(LONG +/- X) TO FOOTING (HORIZ +/- Y) TO FOOTING

ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 2			
Dead Load	0.378 Kips	0.000 Kips	0.000 Kips
Wind Load as Inward Pressure	0.000 Kips	0.000 Kips	3.864 Kips
Wind Load as Outward Pressure	0.000 Kips	0.000 Kips	-4.294 Kips
Column 3			
Dead Load	0.378 Kips	0.000 Kips	0.000 Kips
Wind Load as Inward Pressure	0.000 Kips	0.000 Kips	3.864 Kips
Wind Load as Outward Pressure	0.000 Kips	0.000 Kips	-4.294 Kips

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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

)ESIGN DATA ENDWALL(s): EWD

Column 1 (Cee)

Anchor Rods:	4	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

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Column 2 (Hot Rolled)

Anchor Rods:	4	Base Plate Width:	8.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.13"
Column Depth:	8.11"	Base Plate Thickness:	0.38"
Flange Width:	4.02"		

Column 3 (Hot Rolled)

Anchor Rods:	4	Base Plate Width:	8.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.13"
Column Depth:	8.11"	Base Plate Thickness:	0.38"
Flange Width:	4.02"		

Column 4 (Cee)

Anchor Rods:	4	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

(LONG +/- X) TO FTG (HORIZ +/- Y) TO FTG

ual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1			
Collateral Load	0.573 Kips	0.000 Kips	0.018 Kips
Dead Load	0.505 Kips	0.000 Kips	0.013 Kips
Live Load	2.293 Kips	0.000 Kips	0.071 Kips
Seismic Force Left	0.000 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	0.000 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-3.078 Kips	0.000 Kips	-0.095 Kips
Wind Force Right	-3.078 Kips	0.000 Kips	-0.095 Kips
Wind Load as Inward Pressure	-3.078 Kips	0.000 Kips	-0.095 Kips
Wind Load as Outward Pressure	-3.078 Kips	0.000 Kips	-0.095 Kips
Column 2			
Collateral Load	1.468 Kips	0.000 Kips	-0.039 Kips
Dead Load	1.463 Kips	0.000 Kips	-0.029 Kips
Live Load	5.873 Kips	0.000 Kips	-0.156 Kips
Seismic Force Left	-1.087 Kips	-0.980 Kips	0.000 Kips
Seismic Force Right	1.087 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-7.996 Kips	-1.869 Kips	0.158 Kips
Wind Force Right	-3.850 Kips	0.000 Kips	0.158 Kips
Wind Load as Inward Pressure	-5.923 Kips	0.000 Kips	4.022 Kips
Wind Load as Outward Pressure	-5.923 Kips	0.000 Kips	-4.136 Kips
Column 3			
Collateral Load	1.468 Kips	0.000 Kips	-0.039 Kips
Dead Load	1.463 Kips	0.000 Kips	-0.029 Kips
Live Load	5.873 Kips	0.000 Kips	-0.156 Kips
Seismic Force Left	1.087 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	-1.087 Kips	0.980 Kips	0.000 Kips
Wind Force Left	-3.850 Kips	0.000 Kips	0.158 Kips
Wind Force Right	-7.996 Kips	1.869 Kips	0.158 Kips
·d Load as Inward Pressure	-5.923 Kips	0.000 Kips	4.022 Kips
d Load as Outward Pressure	-5.923 Kips	0.000 Kips	-4.136 Kips
Column 4			
Collateral Load	0.573 Kips	0.000 Kips	-0.018 Kips

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ıdividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 4			
Dead Load	0.505 Kips	0.000 Kips	-0.013 Kips
Live Load	2.293 Kips	0.000 Kips	-0.071 Kips
Seismic Force Left	0.000 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	0.000 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-3.078 Kips	0.000 Kips	0.095 Kips
Wind Force Right	-3.078 Kips	0.000 Kips	0.095 Kips
Wind Load as Inward Pressure	-3.078 Kips	0.000 Kips	0.095 Kips
Wind Load as Outward Pressure	-3.078 Kips	0.000 Kips	0.095 Kips

#	Rafter Type	Rafter Depth
1	Hot Rolled	9.66"
2	Hot Rolled	9.66"

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Project ID: PUBLIC WORKS BLDG

)esign Notes

Buyer is responsible for selecting the appropriate thermal blocks and clips for standing seam roofs for use with the insulation used on the project.
Buyer is responsible for determining the correct fastener length for use with the insulation used on the project.
See the Help file or contact the Manufacturer for documents regarding the proper selection of fasteners, clips and thermal blocks.

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Calculations

DLR Group

Date	10/21/09
Project Name	Gila County Sign Shop
Project No.	30-09119-00
Subject	Foundations
Computed	EDN
Checked	
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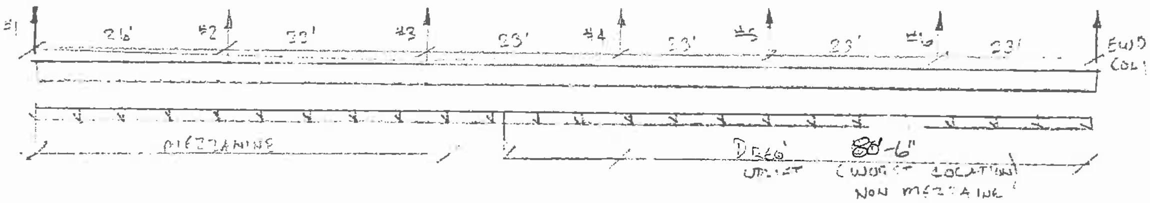
WF 1

SWA (Column 2)

* Vertical uplift : Longitudinal Primary Wind Load 2 Load case
* Collateral load not included

Load combo :

$\boxed{0.9D + 1.6W}$

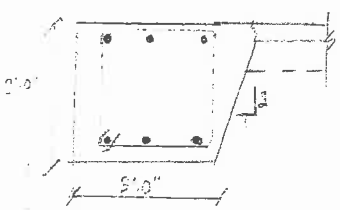


Factored uplift : $0.9D_{REQ'D} = \dfrac{0.9(0.5+3.3+3.3+3.3) - 1.6(13.0+13.0+12.9+3.1)}{80'\text{-}6"} = -0.80\text{KLF}$

Unfactored uplift : $0.6D_{REQ'D} = \dfrac{0.6(0.5+3.3+3.3+3.3) - 1.0(13.0+13.0+12.9+3.1)}{80'\text{-}6"} = -0.74\text{KLF}$

$\boxed{0.6D + W}$

Ftg size : Try 2'-0" deep x 3'-0" wide



W = .15 kcf (2 x 3 + 0.5(2 x 1)) = 1.02 KLF

W > W REQ'D ∴ 2 x 3 footing OK for uplift, FS = 1.02 / 0.74 = 1.4 ≥ 1.5

SOG not included for resistance.

Size OK for uplift

Footing design for uplift not computed since downward forces will control design.

D2

Calculations

DLR Group

Date	10/21/09
Project Name	Gila County Sign Shop
Project No.	32-07118-00
Subject	Foundations
Computed	EDN
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WF 1 SWA (column 2)

DOWNWARD: LATERAL PRIMARY WIND LOAD 2 LOAD CASE FOR WIND.

LOAD COMBO: * COLLATERAL LOAD INCLUDED AS DEAD LOAD

① 1.2D + 1.6L + 0.5Lr
② 1.2D + 1.0L + 1.6Lr

D = 16.4K, L = 21.9K, Lr = 9.6K, W = -3.4K

D = 29.1K, L = 40.5K, Lr = 16.3K, W = -3.1K

D = 27.1K, L = 18.6K, Lr = 15.8K, W = -3.0K

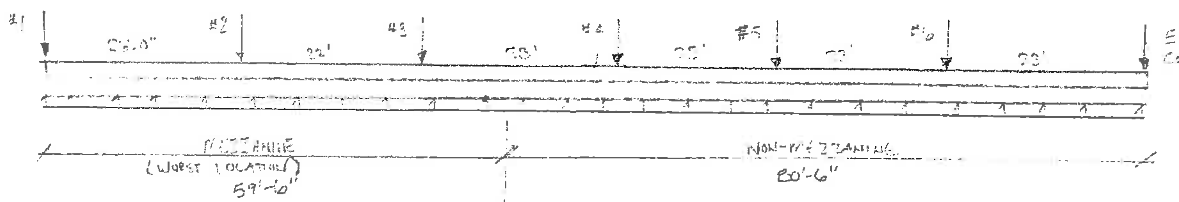
D = 7.3K, L = 0, Lr = 16.1K, W = -3.0K

D = 7.3K, L = 0, Lr = 16.1K, W = -3.0K

D = 7.3K, L = 0, Lr = 16.0K, W = -3.0K

D = 1.1K, L = 0, Lr = 2.3K, W = -3.1K

q_u =



① #1 P_u = 1.2(16.4) + 1.6(21.9) + 0.5(9.6) = 59.5K
① #2 P_u = 1.2(29.1) + 1.6(40.5) + 0.5(16.3) = 107.9K
① #3 P_u = 1.2(27.1) + 1.6(18.6) + 0.5(15.8) = 70.2K

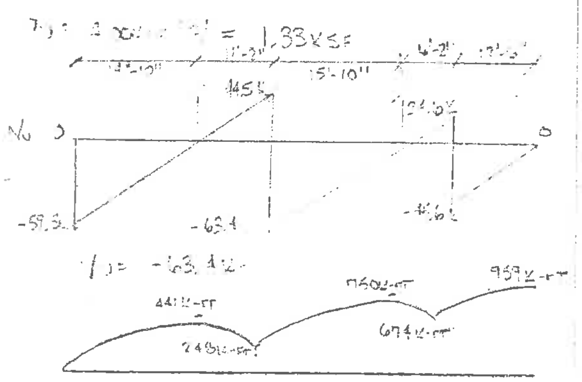
② #4 P_u = 1.2(7.3) + 1.6(16.1) = 34.5K
② #5 P_u = 1.2(7.3) + 1.6(16.1) = 34.6K
② #6 P_u = 1.2(7.3) + 1.6(16.0) = 34.6K
② #7 P_u = 1.2(1.1) + 1.6(2.3) = 5.0K

$q_u = \frac{(59.5 + 107.9 + 70.2)}{59'\text{-}6"}$ = 4.00KLF

$q_u = \frac{(34.5 + 34.6 + 34.6 + 5.0)}{80'\text{-}6"}$ = 1.35KLF

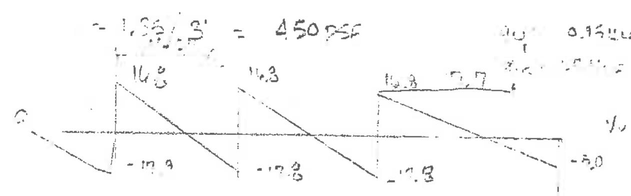
q_u = 4.00KLF / 3' = 1.33KSF

~1.35 / 3' = 450 PSF



V_u = -63.4K

M_u = 959K-FT

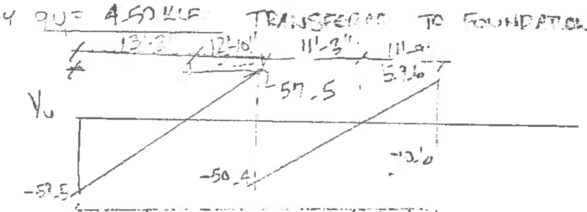


V_u = -17.9 ≈ 18.0K

M_u = 126.2K-FT

FOR MORE BALANCED MOMENT / SHEAR

SAY q_u = 4.50KLF TRANSFERRED TO FOUNDATION



q_u

V_u = -54.0K

M_u = 392 K-FT

q_u = 4.50KLF

q = 4.50/1.5 = 3.0KLF ≈ 1000PSF < 2500PSF OK

Calculations

DLR Group

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WF1 (CONT)

FOOTING DESIGN : NON MEZZANINE. ϕV_c : THE CONCRETE BEAM CAPACITY TABLE

TRY 2'x 3' $b = 36"$ $d = 24 - 3 - 3/8 = 20.625"$ ϕM_n

V_u = [illegible]8.0k

ϕV_c = 5[illegible].6k $\frac{1}{2}\phi V_c > V_u \Rightarrow$ NO SHEAR STEEL REQ'D

M_u = 1[illegible].2k-FT

ϕM_n = 1[illegible]2.9k-FT

5-#6 REBAR @ TOP & BOTTOM FOR FLEXURE
#4 STIRRUP @ 13" OC FOR SUPPORT

2'x3' FOOTING

FOOTING DESIGN : MEZZANINE TRY 3'x3' $b = 36$ $d = 36 - 3 - 7/16 = 32.56"$

V_u = 54.0k

ϕV_c = [illegible]5.9k $\frac{1}{2}\phi V_c < V_u \Rightarrow$ SHEAR STEEL REQ'D #4 @ 12" OC. ϕV_n = 142.3k $\phi V_n > V_u$

M_u = 392 K-FT

ϕM_n = 421.5k-FT

5-#7 REBAR @ TOP & BOTTOM FLEXURE
#4 STIRRUP @ 12" OC FOR SHEAR

3'x3' FOOTING.

D4

DLR Group

Calculations

Date

Project Name

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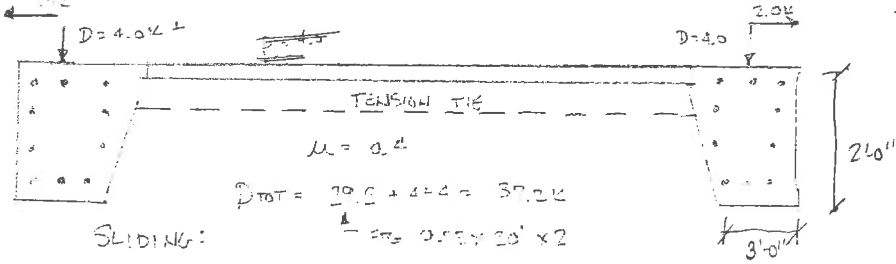
WF 1 HORIZONTAL LOAD & SLIDING

DESIGN HORIZONTAL LOAD OF FOOTING & SLIDING RESISTANCE.

HORIZ: . W ← 2.0k 2.0k → = 5.7k ≅ 6.0k / FRAME

VERT:



$\mu = 0.4$

$D_{TOT} = 29.2 + 4 + 4 = 37.2k$

$\uparrow$ FTG 0.15 × 20' × 2

SLIDING:

$H_{RESIST} = 0.4(37.2) = 14.9k$

$FS = 14.9 / 6 = 2.5 > 1.5$ SLIDING OK → PASSIVE PRESSURE NOT REQ'D.

OVERTURN → WAS VERIFIED W/ UPLIFT CALCS.

BRG PRESSURE → UNIFORM LOAD UNDER FOOTING DESIGNED W/ FOOTING.

DS

Calculations

DLR Group

Date	10/21/09
Project Name	Gila Sign Chse
Project No.	50-09118-00
Subject	Foundations
Computed	EDM
Checked	
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WF2   EWB: ROOF LOAD GOES TO COLUMNS ON FRAME #1 (⊥ FOOTINGS)

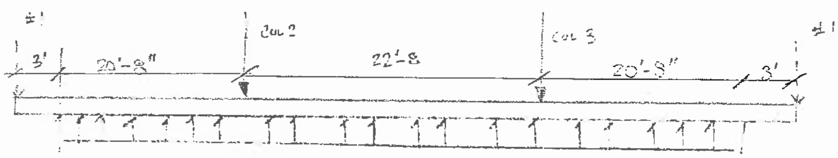
DOWNWARD:

LOAD COMBO

$\boxed{1.2D + 1.6L + 0.5L_r}$

D = 23.8 K
L = 36.6 K
L_r = 0

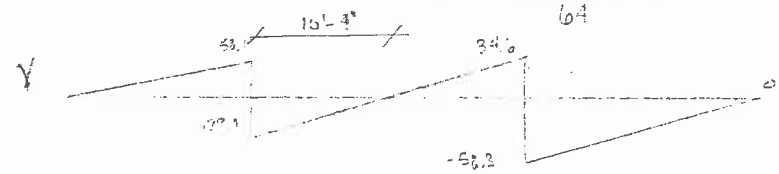
D = 23.8 K
L = 40.2 K
L_r = 0



COL 2 P_u = 1.2(23.8) + 1.6(36.6) = 87.1 K

COL 3 P_u = 1.2(23.5) + 1.6(40.2) = 92.9 K

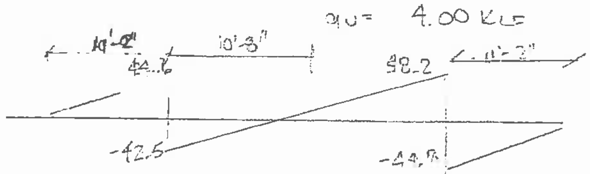
$q_u = \frac{(87.1 + 92.9)}{64} = 2.81$ KLF / 3 = 0.94 KSF



V_u = 58.5K

M_u = 164.0 K-FT   TRY FOR MORE BALANCED SHEAR & MOMENT BY CHANGING SOIL PRESSURE

q_u = 4.00 KLF



V_u = 48.2

M_u = 311.5 K-FT

FOOTING DESIGN   TRY 3x3

ϕV_n = 142.3 K > V_u (OK)

ϕM_n = 421.3 K-FT > M_u (OK)

$\boxed{\text{5-\#7 REBAR TOP \& BOTTOM w/ \#4 @ 12" O.C. STIRRUPS}}$

3'x3' FOOTING

D6

DLR Group

Calculations

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Project Name
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WF3 EWD: NO MEZZANINE LOAD, COL 1 & 4 LOADS TO 1 FOOTINGS, COLLATERAL NOT INCLUDED.

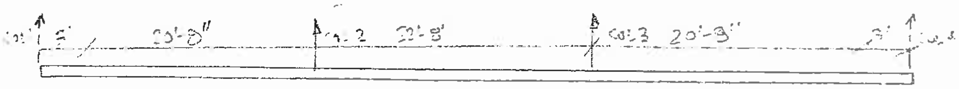
UPWARD ↑:

$0.9D + 1.6W$

$0.6D + W$

D = 1.5
W = -8.0

D = 1.5
W = -8.0



UNFACTORED UPLIFT: $0.6 D_{REQ'D} = \frac{0.6(1.5+1.5) - 1.0(8.0+8.0)}{(20.66+20.66+22.66)} = -0.37$ KLF

$FS = \frac{0.73}{0.37} = 2.0 > 1.5$ (OK)

FTG SIZE: TRY 3'-0" DEEP × 2'-0" WIDE

$W = 145(3\times2 + 0.5(2\times1)) = 0.73$ KLF

W > WREQ'D SIZE OK FOR UPLIFT

$\frac{wl^2}{2} = \frac{.073(20.66)^2}{2} = 15.6$ K-FT

FOOTING DESIGNED FOR DOWNWARD LOAD WHICH CONTROLS.

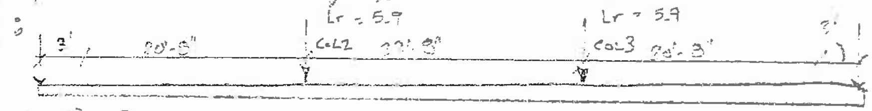
* COLLATERAL INCLUDED

DOWNWARD ↓:

$1.2D + 1.6Lr$

D = 3.0
Lr = 5.9

D = 3.0
Lr = 5.9



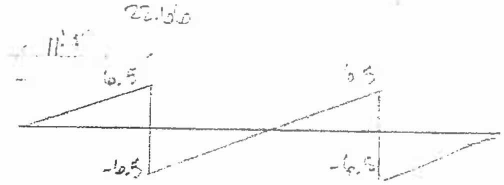
COL 2 $R = 1.2(3.0) + 1.6(5.9) = 13.0$ K

COL 3 $R_u = 1.2(3.0) + 1.6(5.9) = 13.0$ K

$q_u = \frac{(13.0+13.0)}{64'} = 0.41$ K/F / 2 = 205 PSF

FOR MORE BALANCED MOMENT / SHEAR INCREASE BRG PRESSURE TO FOUNDATION OVER SHORTER LENGTH

$q_u = \frac{13}{22.66} = 0.57$ KLF / 2 = 287 PSF



$V_u = 6.5$ K

$M_u = 36.9$ K-FT

FOOTING DESIGN:

$\phi V_n = 39.7$ K ∴ $\frac{1}{2}\phi V_n > V_u$ NO SHEAR REINF REQ'D

$\phi M_n = 153.1$ K-FT > M_u

4-#6 REBAR TOP & BOTTOM FLEXURAL
#4 STIRRUPS @ 48" O.C. AT SUPPORT

2x2 FOOTING

Bentley DLR Group

Current Date: 10/20/2009 7 10 PM
Units system: English
File name: L:\Projects\30-09108-00\+Design\3\SF1 .fdt

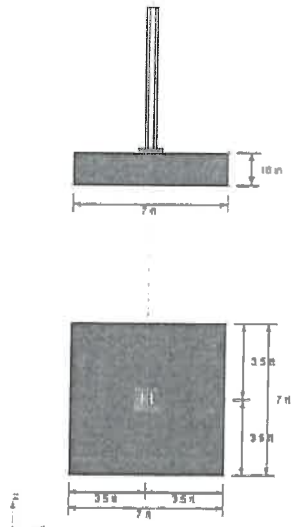
Design Results

Reinforced Concrete Footings

GENERAL INFORMATION

Global status	OK
Design Code	ACI 318-05
Footing type	Spread
Column type	Steel

Geometry



Length	7 00 [ft]
Width	7 00 [ft]
Thickness	1 50 [ft]
Base depth	2 00 [ft]
Base area	49 00 [ft2]
Footing volume	73.50 [ft3]
Base plate length	14 00 [in]
Base plate width	14 00 [in]
Column length	6 00 [in]
Column width	6 00 [in]

Column location relative to footing g.c. Centered

Materials

Concrete, f'c	3.00 [Kip/in2]	Steel, fy	60 00 [Kip/in2]
Concrete type	Normal	Epoxy coated	No
Concrete elasticity modulus	3122 02 [Kip/in2]	Steel elasticity modulus	29000 00 [Kip/in2]
Unit weight	0 15 [Kip/ft3]		

Soil

Modulus of subgrade reaction	200 00 [Kip/ft3]	Cohesion	0 00 [Kip/in2]
Unit weight (wet)	0 11 [Kip/ft3]	Internal friction angle	30 00 [°]
Saturated unit weight	0 14 [Kip/ft3]	Depth of water level	100 00 [ft]
Slope of ground from base	0.00 [°]		

Footing reinforcement

Free cover	3 00 [in]
Maximum Rho/Rho balanced ratio	: 0.75
Bottom reinforcement // to L (xx)	8-#6 @ 11.00"
Bottom reinforcement // to B (zz)	8-#6 @ 11 00" (Zone 1)

Load conditions to be included in design

Service loads:

SC1	DL

Design strength loads

DC1	1 4DL
DC2	1 2DL+1 6LL

Loads

Condition	Axial [Kip]	Mxx [Kip*ft]	Mzz [Kip*ft]	Vx [Kip]	Vz [Kip]
DL	40 00	0.00	0 00	0.00	0 00
LL	75 00	0.00	0 00	0.00	0 00

RESULTS:

Status : OK

Soil Foundation Interaction

Soil stress due to the footing self weight and su...	0 00205 [Kip/in2]
Min. safety factor for bearing capacity	2 50
Min. safety factor for sliding	1 25
Min. safety factor for overturning	1 25

Initial effective stress (qef)	: 0 00153 [Kip/in2]
Controlling condition	: SC1

Condition	qu [Kip/in2]	qnmax [Kip/in2]	qnav [Kip/in2]	FS c.p.	Δmax [in]	Area comp(%)	Overturning FSx	Overturning FSz	FS slip
SC1	0 0174	0.00608	0 00608	2 80	0.0656	100	1000.00	1000 00	1000.00

Bending

Factor ϕ	0 90
Min rebar ratio	: 0 00180

Development length

Axis	Pos	ld [in]	lhd [in]	Dist1 [in]	Dist2 [in]
zz	Bot	17.39	8 06	34 00	34 00
xx	Bot	16.40	8 00	34 00	34 00

Axis	Pos.	Condition	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Asreq [in2]	Asprov [in2]	Asreq/Asprov	Mu/(ϕ*Mn)
zz	Top	DC1	0.00	0 00	0.00	0.00	0 000	0 000
zz	Bot	DC2	114 08	223 85	2.72	3 52	0 773	0 510
xx	Top	DC1	0.00	0.00	0.00	0.00	0.000	0.000
xx	Bot	DC2	114 08	211 97	2.72	3.52	0 773	0 538

Shear

Factor ϕ	: 0 75
Shear area (plane zz)	: 6 53 [ft2]
Shear area (plane xx)	: 6 09 [ft2]

Plane	Condition	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
xy	DC2	46 25	127 67	0 483
yz	DC2	44.75	134.58	0 443

Punching shear

Perimeter of critical section (b...	8 08 [ft]
Punching shear area	: 9 90 [ft2]

Column	Condition	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
column 1	DC2	164 00	302 84	0.078

Notes

* Soil under the footing is considered elastic and homogeneous. A linear soil pressure variation is assumed
* The bending reinforcement considers the minimum reinforcement ratio given by the Code
* The design bending moment is calculated at the critical sections located at the support faces
* Only rectangular footings with uniform sections and rectangular columns are considered.
* The nominal shear strength is calculated in critical sections located at a distance d from the support face
* The punching shear strength is calculated in a perimetral section located at a distance d/2 from the support faces
* Transverse reinforcement is not considered on footings
* Values shown in red are not in compliance with a provision of the code

*qef = Initial effective stress at foundation level prior to loading.

*qu = Meyerhof, Hansen or Vesic ultimate soil bearing capacity.

*qnmax = Maximum pressure at foundation base. qnmax=qmax-qef

SF (bearing) = Safety factor for bearing capacity, SF= (qu-qef)/qnmax

SF (sliding) = Safety factor for sliding

*Δmax = maximum total settlement (considering an elastic soil modeled by the subgrade reaction modulus)

* Mn = Nominal moment strength.

* Mu/(ϕ*Mn) = Strength ratio.

* Vn = Nominal shear or puncture force (for footings Vn=Vc)

* Vu/(ϕ*Vn) = Shear or punching shear strength ratio

D8

Calculations

DLR Group

Date 10/2/09
Project Name Gila Stem Shop
Project No. 30-0?18-00
Subject Foundations
Computed
Checked
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SF1

$P_D = 43.2K$
$P_L = 74.3K$
117.5K ⇒ 7x7x 1'-6" DEEP +6" (blockout) = 2'-0" DEEP — E 2500 PSF
7-#6 BARS
8'-8' — F 3000 PSF

Column by DEMB / Prelim size
KL = 12'-0"
$P_u = 1.2(43.2) + 1.6(74.3) = 170.7K$
$\phi P_n = 205K$
$\phi P_n > P_u$
* HSS 6x6x5/16" OK
* Does not account for building drift moment.

SF2

$P_D = 26.0K$
$P_L = 41.4K$
67.4K ⇒ 6x6 x 1'-6" DEEP +6" blockout ≈ 2'-0" DEEP — D 2500 PSF
5-#6 BARS — D 2000 PSF

SF3

$P_D = 4.1K$
$P_L = 6.5K$
10.6K ⇒ 3x3x 1'-0" +6" blockout ≈ 1'-6" DEEP — A 2500 PSF
A 2000 PSF

Calculations

DLR Group

~~840~~
D9

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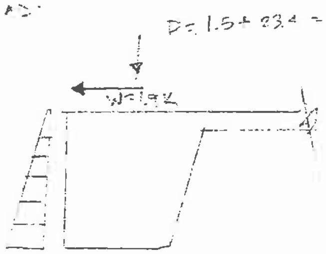
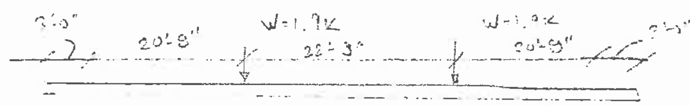
WF3

HORIZONTAL LOAD:

D = 1.5 + 23.4 = 24.9K





$q_{= }$ ~~0.5~~ 240pcf/ft

$q_{need} = \frac{(1.9+1.9)}{(20.6+20.6+22.(...))} = 38\,PLF$

$q > q_{needed}$

Sliding: $0.4(24.9) = 10.0^K > W$

$FS = 10/1.9 = 5.26$ (OK)

Passive pressure [illegible]

Project Gila Country Sign Shop
Subjec ANCHOR DESIGN ACI 318-05 APPENDIX D
Date: 10/28/2009

Project Number: 30-09118-00
Computed by: EDN
Page:

Vertical (Tension)

Gridline	Group Type	D	Dc	L	Lr/S	W1	W2	W3	W4	W5	W6	W7	W8	E
A1	Frame1, Col2	14	2.4	21.9	9.6	-6.2	-3.4	-9.5	-6.7	-7	-7.8	-4.4	-4.7	0.4
D1	Frame1, Col1	14	2.4	21.9	9.6	-9.5	-6.7	-6.2	-3.4	-7.8	-7	-4.7	-4.4	-0.4
A2	Frame2, Col2	24.9	4.1	40.5	16.3	-7.8	-3.1	-11.4	-6.6	-11.8	-13.1	-7.4	-8	0.6
D2	Frame2, Col1	24.9	4.1	33.8	16.3	-11.4	-6.6	-7.8	-3.1	-13.1	-11.8	-8	-7.4	-0.6
A3	Frame3, Col2	13.2	3.9	18.6	15.8	-7.6	-3	-11	-6.4	-11.4	-12.7	-7.2	-7.7	0.6
D3	Frame3, Col1	13.2	3.9	15.5	15.8	-11	-6.4	-7.6	-3	-15.5	-14.2	-10.5	-10	-3.5
A4	Frame4, Col2	3.3	4	0	16.1	-7.8	-3	-11.3	-6.5	-11.7	-13	-7.3	-7.8	0.6
D4	Frame4, Col1	3.3	4	0	16.1	-11.3	-6.5	-7.8	-7.3	-15.8	-14.5	-10.7	-10.1	-3.5
A5	Frame5, Col2	3.3	4	0	16.1	-7.8	-3	-11.3	-6.5	-11.7	-13	-7.3	-7.9	0.6
D5	Frame5, Col1	3.3	4	0	16.1	-11.3	-6.5	-7.8	-3	-15.8	-14.5	-10.7	-10.1	-3.5
A6	Frame6, Col2	3.3	4	0	16	-7.7	-3	-11.2	-6.5	-11.6	-12.9	-7.3	-7.8	0.6
D6	Frame6, Col1	3.3	4	0	16	-11.2	-6.5	-7.7	-3	-1.9	-11.6	-7.8	-7.3	-0.6
B1	EWB, Col3	0.4	0	0	0	0	0	0	0	0	0	0	0	
C1	EWB, Col2	0.4	0	0	0	0	0	0	0	0	0	0	0	-1.1
B7	EWD, Col2	1.5	1.5	0	5.9	-8	-3.9	0	0	-5.9	-5.9	0	0	-1.1
C7	EWD, Col3	1.5	1.5	0	5.9	-8	-3.9	0	0	-5.9	-5.9	0	0	-1.1
A.1/4.1	Portal	0	0	0	0	6.7	6.7	6.7	6.7	-5.6	-5.6	-5.6	-5.6	-5.6
A.1/4.9	Portal	0	0	0	0	6.7	6.7	6.7	6.7	-5.6	-5.6	-5.6	-5.6	-5.6

Horizontal (Shear +/- y) to Footing

Gridline	Group Type	D	Dc	L	Lr/S	W1	W2	W3	W4	W5	W6	W7	W8	E
A1	Frame1, Col2	-1.1	-1.3	0	-5.3	1.5	0.9	6.6	6	2.2	2	0.7	0.7	-0.7
D1	Frame1, Col1	-1.1	-1.3	0	-5.3	6.6	6	1.5	0.9	2	2.2	0.7	0.7	0.7
A2	Frame2, Col2	-1.7	-2.4	0	-9.4	2	0.9	7.7	6.7	4	3.7	1.4	1.3	-1.1
D2	Frame2, Col1	-1.7	-2.4	0	-9.4	7.7	6.7	2	0.9	3.7	4	1.3	1.4	1.1
A3	Frame3, Col2	-1.6	-2.3	0	-9.1	1.9	0.9	7.5	6.5	3.9	3.6	1.3	1.2	-1
D3	Frame3, Col1	-1.6	-2.3	0	-9.1	7.5	6.5	1.9	0.9	3.6	3.9	1.2	1.3	1
A4	Frame4, Col2	-1.7	-2.3	0	-9.3	2	1	7.7	6.6	3.9	3.6	1.4	1.3	-1
D4	Frame4, Col1	-1.7	-2.3	0	-9.3	7.7	6.6	2	0.9	3.6	3.9	1.3	1.4	1
A5	Frame5, Col2	-1.7	-2.3	0	-9.3	2	1	7.7	6.6	3.9	3.6	1.4	1.3	-1
D5	Frame5, Col1	-1.7	-2.3	0	-9.3	7.7	6.6	2	0.9	3.6	3.9	1.3	1.4	1
A6	Frame6, Col2	-1.7	-2.3	0	-9.3	1.9	0.9	7.6	6.6	3.9	3.6	1.4	1.2	-1
D6	Frame6, Col1	-1.7	-2.3	0	-9.3	7.6	6.6	2	0.9	3.6	3.9	1.2	1.4	1
B1	EWB, Col3	0	0	0	0	-3.9	-3.9	-3.9	-3.9	4.3	4.3	4.3	4.3	0
C1	EWB, Col2	0	0	0	0	-3.9	-3.9	-3.9	-3.9	4.3	4.3	4.3	4.3	0
B7	EWD, Col2	0	0	0	0.2	-4	-4	-4	-4	4.2	4.2	4.2	4.2	0
C7	EWD, Col3	0	0	0	0.2	-4	-4	-4	-4	4.2	4.2	4.2	4.2	0
A.1/4.1	Portal	0	0	0	0	0	0	0	0	0	0	0	0	0
A.1/4.9	Portal	0	0	0	0	0	0	0	0	0	0	0	0	0

Longitudnal (Shear +/- x) to Footing

Gridline	Group Type	D	Dc	L	Lr/S	W1	W2	W3	W4	W5	W6	W7	W8	E
A1	Frame1, Col2	0	0	0	0	0	0	0	0	0	0	0	0	0
D1	Frame1, Col1	0	0	0	0	0	0	0	0	0	0	0	0	0
A2	Frame2, Col2	0	0	0	0	0	0	0	0	0	0	0	0	0
D2	Frame2, Col1	0	0	0	0	0	0	0	0	0	0	0	0	0
A3	Frame3, Col2	0	0	0	0	0	0	0	0	0	0	0	0	0
D3	Frame3, Col1	0	0	0	0	0	0	0	0	2.9	2.9	2.9	2.9	16
A4	Frame4, Col2	0	0	0	0	0	0	0	0	0	0	0	0	0
D4	Frame4, Col1	0	0	0	0	0	0	0	0	2.9	2.9	2.9	2.9	16
A5	Frame5, Col2	0	0	0	0	0	0	0	0	0	0	0	0	0
D5	Frame5, Col1	0	0	0	0	0	0	0	0	2.9	2.9	2.9	2.9	16
A6	Frame6, Col2	0	0	0	0	0	0	0	0	0	0	0	0	0
D6	Frame6, Col1	0	0	0	0	0	0	0	0	0	0	0	0	0
B1	EWB, Col3	0	0	0	0	0	0	0	0	0	0	0	0	0
C1	EWB, Col2	0	0	0	0	0	0	0	0	0	0	0	0	0
B7	EWD, Col2	0	0	0	0	1.9	1.9	1.9	1.9	0	0	0	0	1
C7	EWD, Col3	0	0	0	0	-1.9	-1.9	-1.9	-1.9	0	0	0	0	-1
A.1/4.1	Portal	0	0	0	0	-3	-3	-3	-3	3.1	3.1	[illegible]	[illegible]	[illegible]
A.1/4.9	Portal	0	0	0	0	3	3	3	3	-3.1	-3.1	[illegible]	[illegible]	[illegible]

Anchor Bolt Combo's

Gridline	Group Type	Vertical Load Combo: Nu max 1	Nu max 2	Nu max 3	Nu min 4	Nu min 5	Horiz Load Combo: -Vuy 1	-Vuy 2	-Vuy 3	+Vuy 4	+Vuy 5	Long Load Combo (Vux): -Vux 1	-Vux 2	-Vux 3	+Vux 4	+Vux 5
	Frame1, Col2	59.5	56.9	40.9	-2.6	13.0	-5.5	-11.4	-4.4	9.6	-1.7	0.0	0.0	0.0	0.0	0.0
D1	Frame1, Col1	59.5	56.9	40.9	-2.6	12.2	-5.5	-11.4	-4.4	9.6	-0.3	0.0	0.0	0.0	0.0	0.0
A2	Frame2, Col2	107.8	101.4	78.5	1.5	23.0	-9.6	-20.0	-8.2	10.8	-2.6	0.0	0.0	0.0	0.0	0.0
D2	Frame2, Col1	97.0	94.7	71.8	1.5	21.8	-9.6	-20.0	-8.2	10.8	-0.4	0.0	0.0	0.0	0.0	0.0
A3	Frame3, Col2	58.2	64.4	42.2	-8.4	12.5	-9.2	-19.2	-7.8	10.6	-2.4	0.0	0.0	0.0	0.0	0.0
D3	Frame3, Col1	53.2	61.3	39.1	-12.9	8.4	-9.2	-19.2	-7.8	10.6	-0.4	0.0	0.0	0.0	4.6	16.0
A4	Frame4, Col2	16.8	34.5	12.0	-17.8	3.6	-9.5	-19.7	-7.9	10.8	-2.5	0.0	0.0	0.0	0.0	0.0
D4	Frame4, Col1	16.8	34.5	6.4	-22.3	-0.5	-9.5	-19.7	-8.0	10.8	-0.5	0.0	0.0	0.0	4.6	16.0
A5	Frame5, Col2	16.8	34.5	12.0	-17.8	3.6	-9.5	-19.7	-7.9	10.8	-2.5	0.0	0.0	0.0	0.0	0.0
D5	Frame5, Col1	16.8	34.5	12.0	-22.3	-0.5	-9.5	-19.7	-8.0	10.8	-0.5	0.0	0.0	0.0	4.6	16.0
A6	Frame6, Col2	16.8	34.4	12.0	-17.7	3.6	-9.5	-19.7	-8.0	10.6	-2.5	0.0	0.0	0.0	0.0	0.0
D6	Frame6, Col1	16.8	34.4	13.7	-15.6	2.4	-9.5	-19.7	-8.0	10.6	-0.5	0.0	0.0	0.0	0.0	0.0
B1	EWB, Col3	0.5	0.5	0.5	0.4	0.4	0.0	-3.1	-6.2	6.9	0.0	0.0	0.0	0.0	0.0	0.0
C1	EWB, Col2	0.5	0.5	0.5	0.4	-0.7	0.0	-3.1	-6.2	6.9	0.0	0.0	0.0	0.0	0.0	0.0
B7	EWD, Col2	6.6	13.0	6.6	-11.5	0.3	0.1	-2.9	-6.3	6.7	0.0	0.0	0.0	0.0	3.0	1.0
C7	EWD, Col3	6.6	13.0	6.6	-11.5	0.3	0.1	-2.9	-6.3	6.7	0.0	0.0	-1.5	-3.0	0.0	-1.0
A.1/4.1	Portal	0.0	5.4	10.7	-9.0	-5.6	0.0	0.0	0.0	0.0	0.0	0.0	-2.4	-4.8	5.0	3.1
A.1/4.9	Portal	0.0	5.4	10.7	-9.0	-5.6	0.0	0.0	0.0	0.0	0.0	0.0	-2.5	-5.0	4.8	-3.1

Gridline	Tension øNmin	Shear y -øVy	+øVy	Shear x -øVx	+øVx	A	B	C	D	E	F	G	H	I	Shear	Tension	Combo
	-27.3	-30.2	5.3	-15.8	15.8	0.38	1.81	0.00	0.00	0.10	0.47	1.90	0.10	0.10			
D1	-27.3	-30.2	5.3	-15.8	15.8	0.38	1.81	0.00	0.00	0.10	0.47	1.90	0.10	0.10			
A2	-27.3	-30.2	5.3	-15.8	15.8	0.66	2.04	0.00	0.00	-0.05	0.61	1.98	-0.05	-0.05			
D2	-27.3	-30.2	5.3	-15.8	15.8	0.66	2.04	0.00	0.00	-0.05	0.61	1.98	-0.05	-0.05			
A3	-27.3	-30.2	5.3	-15.8	15.8	0.64	1.99	0.00	0.00	0.31	0.95	2.30	0.31	0.31			
D3	-27.3	-30.2	5.3	-15.8	15.8	0.64	1.99	0.00	1.01	0.47	1.11	2.47	0.47	1.49			
A4	-27.3	-30.2	5.3	-15.8	15.8	0.65	2.04	0.00	0.00	0.65	1.30	2.69	0.65	0.65			
D4	-27.3	-30.2	5.3	-15.8	15.8	0.65	2.04	0.00	1.01	0.82	1.47	2.85	0.82	1.83			
A5	-27.3	-30.2	5.3	-15.8	15.8	0.65	2.04	0.00	0.00	0.65	1.30	2.69	0.65	0.65			
D5	-27.3	-30.2	5.3	-15.8	15.8	0.65	2.04	0.00	1.01	0.82	1.47	2.85	0.82	1.83			
A6	-27.3	-30.2	5.3	-15.8	15.8	0.65	2.01	0.00	0.00	0.65	1.30	2.65	0.65	0.65			
D6	-27.3	-30.2	5.3	-15.8	15.8	0.65	2.01	0.00	0.00	0.57	1.22	2.58	0.57	0.57			
B1	-16.2	-10.2	4.8	-10.2	10.2	0.61	1.43	0.00	0.00	-0.02	0.59	1.41	-0.02	-0.02			
C1	-16.2	-10.2	4.8	-10.2	10.2	0.61	1.43	0.00	0.00	0.05	0.66	1.48	0.05	0.05			
B7	-16.2	-10.2	4.8	-10.2	10.2	0.62	1.40	0.00	0.30	0.71	1.32	2.11	0.71	1.00			
C7	-16.2	-10.2	4.8	-10.2	10.2	0.62	1.40	0.30	0.00	0.71	1.32	2.11	1.00	0.71			
A.1/4.1	-35.4	-30.2	22.8	-22.4	22.4	0.00	0.00	0.21	0.22	0.25	0.25	0.25	0.47	0.47			
A.1/4.9	-35.4	-30.2	22.8	-22.4	22.4	0.00	0.00	0.22	0.21	0.25	0.25	0.25	0.47	0.47			

Given: Indicates X-bracing loads Indicates Portal Frame loads

Vertical - PEMB Manufacturer
+ is downward load (compression)
- is upward load (tension)

Column 2 - PEMB Manufacturer
+ is toward the concrete edge
- is away from the concrete edge

*Column 1 - PEMB Manufacturer
+ is away from concrete edge
- is toward the concrete edge

*End Wall - PEMB Manufacturer
+ is away from concrete edge
- is toward the concrete edge

Anchor Bolt design
+ is toward the concrete edge (exterior)
- is away from the concrete edge (interior)

[illegible]d on this all the Column 1 signs for shear are switched from the PEMB given loadsto align with anchor bolt design sheets.

*Based on this all the End Wall column signs for shear are switched from PEMB given loads to align with anchor bolt design sheets.

Also the End Wall and Portal frame horizontal and longitudnal loads are switched to align with anchor bolt design direction. for the footing design direction

A	-Vuy / -øVy < 1.0
B	+Vuy / +øVy < 1.0
C	-Vux / -øVx < 1.0
D	+Vux / +øVx < 1.0
E	Nu min / øNmin < 1.0
F	Nu min / øNmin + -Vuy / -øVy < 1.2
G	Nu min / øNmin + Vuy / +øVy < 1.2
H	Nu min / øNmin + -Vux / -øVx < 1.2
I	Nu min / øNmin + +Vux / +øVx < 1.2

Calculations

E2

DLR Group

Date	10/29/09
Project Name	GILA COUNTY SIGN SHOP
Project No.	30-09118-00
Subject	
Computed	EDN
Checked	
Page (of pages)	

AP
GILA
PEF
DAT

ANCHOR DESIGN: (HAIRPIN)

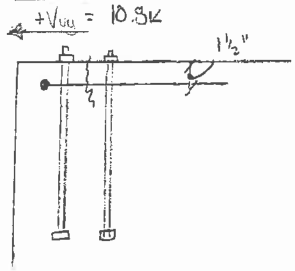
FROM RESULTS $+V_{uy}$ ISSUE ALL COLUMNS EXCEPT PORTAL FRAME, HAIR PIN REBAR REQ'D

FROM RESULTS N_{umin} (TENSION) ALL ANCHOR BOLTS ARE OK W/OUT STIRRUPS.

FROM RESULTS $+V_{ux}$ ARE OK W/ OUT HAIRPIN

FRAME COL

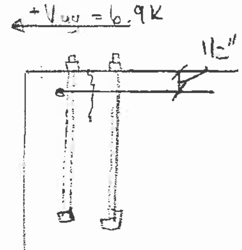
$+V_{uy} = 10.8K$



$\phi V_y = 5.3K$

END WALL COL

$+V_{uy} = 6.9K$



$\phi V_y = 4.8K$

HAIRPIN: #5 REBAR: $A_{vf} = 0.31 \times 2 = 0.62$

TWO BARS IN FRICTION

SHEAR FRICTION: $\phi V_N = \phi A_{vf} F_y (\mu)$

$= 0.75(0.62)(60ksi)(1.4)(1.0)$

$= \underline{39.1K > V_{uy}}$

$\boxed{\text{\#5 HAIR PIN OK FOR ALL LOADING}}$

ALTHOUGH ANCHORS IN TENSION ARE OK PLACE STIRRUP BARS NEXT TO ANCHORS.

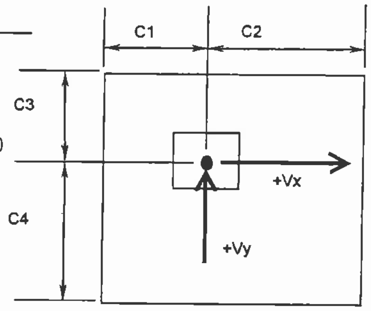
Project: Gila Country Sign Shop
Subject: ANCHOR DESIGN ACI 318-05 APPENDIX D
Date: ########

Project Number: 30-09118-00
Computed by: EDN
Page:

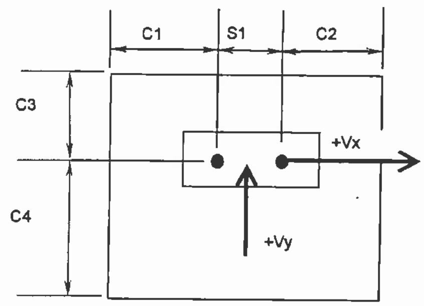
INPUT: Description: Frame Column Bolt Group Loads

No. of Anchors = 4 C1 = 100 IN. S1 = 4 IN.
C2 = 100 IN. S2 = 4 IN.
Anchor Diameter = 0.750 IN. C3 = 4 IN.
C4 = 100 IN.
ANCHOR STEEL DESIGNATION = A307 (A307, A325, A490, or leave blank for ACI design only)
fya = 36,000 YIELD STRENGTH (PSI) PER ASTM F 1554
futa = 58,000 TENSILE STRENGTH (PSI) PER ASTM F 1554
Anchor Embed. = 14 IN. (hef)

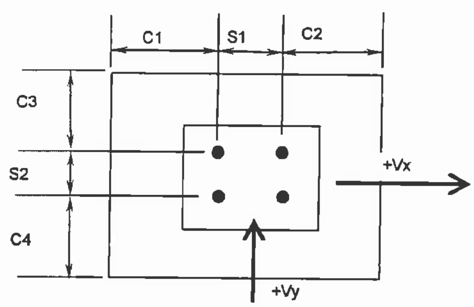
Anchor Type = CI (CI = Cast-In place, PI = Post Installed)
Ductile? DUCTILE DUCTILE or BRITTLE?
Built-Up Grout Pad = NO (YES or NO)
Anchor Head: H (H = headed anchor, J = J or L bolt)
Concrete: NW (NW = Normal weight, LW = All Light weight Concrete, SLW = Sand Light weight Concrete)
f'c = 3,000 PSI (28 Day Compressive Strength)
UNCRACKEI CRACKED or UNCRACKED Concrete
h = 24 IN, (Thickness of Concrete Member)
Condition: B A or B, ("A" applies when supplementary reinf. Is provided, per D.4.4)
Condition for P.I. Anchors = ____ (1, 2, or 3)



ONE ANCHOR



TWO ANCHORS



FOUR ANCHORS

TENSION:

Steel Strength: Nsa = n Ase futa Φns = 0.75

:I 318 0.0 Ase = 0.334 IN^2 futa = 1.9 fya or 125,000 PSI = 58,000 PSI
ΦNsa = 58.1 KIPS
AISC 7-14 ΦNs 59.7 KIPS ΦFt = 33.8 KSI, AISC TABLE 7-14

ΦNsa = 58.1 Kips/GROUP ΦNsa = 14.5 Kips/BOLT

Concrete Break-out Strength: Φcb = 0.70 NW?= 1

ANco = 1764.00 IN^2 (D-6) Anc = 1334.0 IN.(

Nb = 71,267.9 (LB) ACI D-7 & D-8

If three or four edge distances are less than 1.5 hef, (Ca,max<= 1.5 hef) then embedment is limited to the greater of Cmax/1.5, (hef = Ca,max/1.5) or 1/3 of the max spacing between anchors For use in eq. D-4 thru D-11
hef = 14.000 IN.

Ψec,N = 1.00 NO ECCENTRICITY e'nx = 0
e'ny = 0

Ψed,N = 0.76 Modification for edge effects

Ψc,N = 1.25 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψc,N = 1.25 for cast-in anchors
Ψc,N = 1.4 for Post-installed anchors
Ψc,N = 1.0 Cracking, post-installed, or cast-in anchors

ΦNcb = 35.71 KIPS (D-4, D-5)

Pullout Strength in Tension: Φpn = 0.70

Abrg = 0.911 eh= 4.5do for J or L bolts

Np = 21,864.00 LB, D-15, D-16

Ψc,p = 1.4 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψ4 = 1.4 for cast-in anchors
Ψ4 = 1.0 Cracking, cast-in anchors

ΦNpn = 21.43 KIPS, Single Anchor (D-14)

ΦNpng = 85.71 KIPS, Group of Anchors (D-14)

Concrete side-face blowout strength of a headed anchor in tension:

Φnc = 0.7 Not considered when Ca,min > 0.4 hef

Nsb = 39,034.3 C = 4 IN.
C2 = 100 IN.
C2/C = 3

C is equal to the minimum edge distance and C2 is equal to the minimum edge distance perpendicular to C.
If C2 < 3 C then multiply by an additional factor per D.5.4.2.

Φnc = 27.32 KIPS

E7

I-CC
/ED
COMI
G
93-

SHEAR:

Steel Strength of Anchor in Shear: D6.1 Φvsa = 0.65

CI318 0.0, Ase = 0.334 IN² futa = 1.9 fya or 125,000 PSI = 58,000.0 PSI

Post-Installed Anchor Sleeves:
Asl = ______ IN2 futsl = ______ PSI

ΦVsa = 30.2 KIPS, D-18 or D-19
AISC 7-10 ΦVsa = 31.79 KIPS ΦFs = 18 KSI, AISC TABLE 7-10
Assume threads included in shear plane & single shear application

ΦVsa = 30.2 Kips/GROUP	ΦVsa = 7.6 Kips/BOLT

Concrete Breakout Strength of Anchor in Shear, per D.6.2: Φvs = 0.7

Avco+y =	72.0 IN²	Avco+x =	1152.0 IN²
Avco-y =	1152.0 IN²	Avco-x =	1152.0 IN²
Avc+y =	96.0 IN²	Avc+x =	768.0 IN²
Avc-y =	1248.0 IN²	Avc-x =	768.0 IN²
vb+y =	4026.2 LB	vb+x =	32209.7 LB
vb-y =	32209.7 LB	vb-x =	32209.7 LB
Ψec,v +y =	1.0	Ψec,v +x =	1.0
Ψec,v -y =	1.0	Ψec,v -x =	1.0
Ψed+y =	1.0	Ψed+x =	0.8
Ψed-y =	1.0	Ψed-x =	0.8
Ψc,v =	1.4		

Are the Anchors welded to the plate? NO
If yes, then anchor capacity is determined based on the row farthest from the edge and, the center to center spacing of the anchor is not less than 2.5 in.; and supplementary reinforcement is provided at the corners if C2 <= 1.5 hef

If three or more edge distances are less than 1.5 C1, (C2-4 or h<= 1.5 C1) then edge distance C1 is limited to C1 = h/1.5,
For use in eq. D-22 thru D-26

C'1+y =	4.0 IN	C'1+x =	16.0
C'1-Y =	16.0 IN	C'1-x =	16.0

e'vx = 0 IN., Eccentricity of shear force on a group of anchors, X axis
e'vy = 0 IN., Eccentricity of shear force on a group of anchors, Y axis

Ψc,v : D6.2.7, For anchors located in a region of concrete where analysis indicates no cracking (ft < fr) at service loads, then Ψe,v = 1.4

Ψe,v = 1.0, Anchors in cracked concrete with no supplementary reinf.
Ψe,v = 1.2, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & the edge.
Ψe,v = 1.4, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & edge & #4 stirrups.

ΦVcb(g)+y =	5.26 KIPS	ΦVcb(g)+x =	15.78 KIPS
ΦVcb(g) -y =	34.20 KIPS	ΦVcb(g) -x =	15.78 KIPS

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR: D.6.3 :

Φcp = 0.7

Ncb = 51.0 KIPS From D-4 & Above

ΦcpVcp = 71.4 KIPS	D-28

RESULTS:

TENSION KIPS	
ΦNsa =	58.1
ΦNcb =	35.7
ΦNpng =	85.7
Φnc =	27.3
ΦNna =	27.3

SHEAR KIPS			
ΦVsa =	30.2		
ΦVcb(g)+y =	5.26	ΦVcb(g)+x =	15.78
ΦVcb(g) -y =	34.20	ΦVcb(g) -x =	15.78
ΦcpVcp =	71.4		
ΦVn+y =	5.26	ΦVn+x =	15.78
ΦVn - y =	30.22	ΦVn- x =	15.78
HORIZONTAL		LONGITUDNAL	

Project:	Gila Country Sign Shop	Project Number:	30-09118-00
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DES
FIELD
UNITY DEV
911-0
D BY:

SPACING & EDGE DISTANCES:

SPACING:

Minimum center to center spacing of untorqued cast-in-place anchors = 4do= **3.0** IN.

Minimum o.c. spacing of torqued cast-in-place & post-installed anchors = 6do= **4.5** IN.

EDGE DISTANCE:

Minimum edge distance for cast-in headed anchors, un-torqued shall be based on minimum cover requirements for reinforcement of ACI 7.7:

Concrete cast against and permanently exposed to soil: 3 IN.

Concrete exposed to earth or weather:
#6-#18 bars 2 IN.
#5 bar & smaller: 1 1/2 IN.

Concrete not exposed to weather or in contact with ground:
#14 & #18 bars: 1 1/2 IN.
#11 bars & smaller: 3/4 IN.

Minimum edge distance for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in ACI 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 test information, the minimum edge distance shall be taken as not less than:

Undercut anchors, 6do= 4.5 IN.

Torqued-controlled anchors, 8do= 6 IN.

Displacement - controlled anchors, 10do= 7.5 IN.

E6

Project:	Gila Country Sign Shop	Project Number:	30-09118-00
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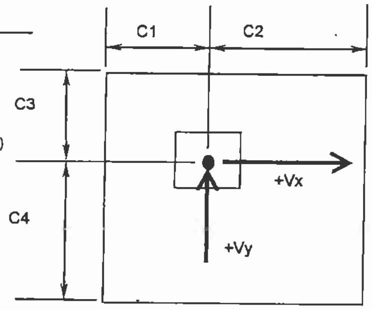
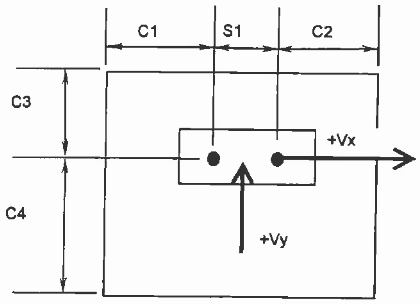
INPUT: Description: End Wall Column Bolt Group Loads

No. of Anchors = 2 C1 = 100 IN. S1 = 4 IN.
C2 = 100 IN. S2 = 0 IN.
Anchor Diameter = 0.625 IN. C3 = 4 IN.
C4 = 20 IN.
ANCHOR STEEL DESIGNATION = A307 (A307, A325, A490, or leave blank for ACI design only)
fya = 36,000 YIELD STRENGTH (PSI) PER ASTM F 1554
futa = 58,000 TENSILE STRENGTH (PSI) PER ASTM F 1554
Anchor Embed. = 8 IN. (hef)

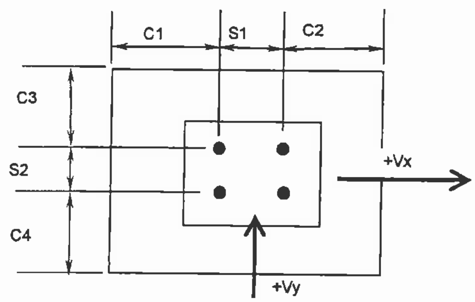
Anchor Type = CI (CI = Cast-In place , PI = Post Installed)
Ductile? DUCTILE DUCTILE or BRITTLE?
Built-Up Grout Pad = NO (YES or NO)
Anchor Head: H (H = headed anchor, J = J or L bolt)
Concrete : NW (NW = Normal weight, LW = All Light weight Concrete, SLW = Sand Light weight Concrete)
f'c = 3,000 PSI (28 Day Compressive Strength)
UNCRACKEI CRACKED or UNCRACKED Concrete
h = 24 IN, (Thickness of Concrete Member)
Condition: B A or B, ("A" applies when supplementary reinf. Is provided, per D.4.4)
Condition for P.I. Anchors = ______ (1, 2, or 3)



ONE ANCHOR



TWO ANCHORS



FOUR ANCHORS

TENSION:

Steel Strength: Nsa = n Ase futa Φns = 0.75

:I 318 0.0 Ase = 0.226 IN^2 futa = 1.9 fya or 125,000 PSI = 58,000 PSI
ΦNsa = 19.7 KIPS
AISC 7-14 ΦNs 20.7 KIPS ΦFt = 33.8 KSI, AISC TABLE 7-14

ΦNsa = 19.7 Kips/GROUP	ΦNsa = 9.8 Kips/BOLT

Concrete Break-out Strength : Φcb = 0.70 NW?= 1

ANco = 576.00 IN^2 (D-6) Anc = 448.0 IN.(

Nb = 29,744.5 (LB) ACI D-7 & D-8

If three or four edge distances are less than 1.5 hef, (Ca,max<= 1.5 hef) then embedment is limited to the greater of Cmax/1.5, (hef = Ca,max/1.5) or 1/3 of the max spacing between anchors For use in eq. D-4 thru D-11
hef = 8.000 IN.

Ψec,N = 1.00 NO ECCENTRICITY e'nx = 0
e'ny = 0

Ψed,N = 0.80 Modification for edge effects

Ψc,N = 1.25 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψc,N = 1.25 for cast-in anchors
Ψc,N = 1.4 for Post-installed anchors
Ψc,N = 1.0 Cracking, post-installed, or cast-in anchors

ΦNcb = 16.19 KIPS	(D-4, D-5)

Pullout Strength in Tension: Φpn = 0.70

Abrg = 0.671 eh= 4.5do for J or L bolts

Np = 16,104.00 LB, D-15, D-16

Ψc,p = 1.4 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψ4 = 1.4 for cast-in anchors
Ψ4 = 1.0 Cracking, cast-in anchors

ΦNpn =	15.78	KIPS, Single Anchor (D-14)
ΦNpng =	31.56	KIPS, Group of Anchors (D-14)

Concrete side-face blowout strength of a headed anchor in tension:

Φnc = 0.7 Not considered when Ca,min > 0.4 hef

Nsb= NOT CONSIDERED

C = 4 IN.
C2 = 100 IN.
C2/C = 3

C is equal to the minimum edge distance and C2 is equal to the minimum edge distance perpendicular to C.
If C2 < 3 C then multiply by an additional factor per D.5.4.2.

Φnc = N/A

SHEAR:

Steel Strength of Anchor in Shear: D6.1 Φvsa = 0.65

CI318 0.0, Ase = 0.226 IN² futa = 1.9 fya or 125,000 PSI = 58,000.0 PSI

Post-Installed Anchor Sleeves:
Asl = ______ IN2 futsl = ______ PSI

ΦVsa = 10.2 KIPS, D-18 or D-19
AISC 7-10 ΦVsa = 11.04 KIPS ΦFs = 18 KSI, AISC TABLE 7-10
Assume threads included in shear plane & single shear application

ΦVsa = 10.2 Kips/GROUP	ΦVsa = 5.1 Kips/BOLT

Concrete Breakout Strength of Anchor in Shear, per D.6.2: Φvs = 0.7

Avco+y = 72.0 IN² Avco+x = 1152.0 IN²
Avco-y = 1800.0 IN² Avco-x = 1152.0 IN²
Avc+y = 96.0 IN² Avc+x = 576.0 IN²
Avc-y = 1536.0 IN² Avc-x = 576.0 IN²

vb+y = 3675.4 LB vb+x = 29403.3 LB
vb-y = 41092.4 LB vb-x = 29403.3 LB
Ψec,v +y = 1.0 Ψec,v +x = 1.0
Ψec,v -y = 1.0 Ψec,v -x = 1.0
Ψed+y = 1.0 Ψed+x = 0.8
Ψed-y = 1.0 Ψed-x = 0.8
Ψc,v = 1.4

Are the Anchors welded to the plate? NO
If yes, then anchor capacity is determined based on the row farthest from the edge and, the center to center spacing of the anchor is not less than 2.5 in.; and supplementary reinforcement is provided at the corners if C2 <= 1.5 hef

If three or more edge distances are less than 1.5 C1, (C2-4 or h<= 1.5 C1) then edge distance C1 is limited to C1=h/1.5,
For use in eq. D-22 thru D-26

C'1+y = 4.0 IN C'1+x = 16.0
C'1-Y = 20.0 IN C'1-x = 16.0

e'vx = 0 IN., Eccentricity of shear force on a group of anchors, X axis
e'vy = 0 IN., Eccentricity of shear force on a group of anchors, Y axis

Ψc,v : D6.2.7, For anchors located in a region of concrete where analysis indicates no cracking (ft < fr) at service loads, then Ψe,v = 1.4

Ψe,v = 1.0, Anchors in cracked concrete with no supplementary reinf.
Ψe,v = 1.2, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & the edge.
Ψe,v = 1.4, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & edge & #4 stirrups.

ΦVcb(g)+y = 4.80	KIPS	ΦVcb(g)+x = 10.81	KIPS
ΦVcb(g) -y = 34.36	KIPS	ΦVcb(g) -x = 10.81	KIPS

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR: D.6.3 : Φcp = 0.7

Ncb = 23.1 KIPS From D-4 & Above

ΦcpVcp = 32.4 KIPS

D-28

RESULTS:

TENSION KIPS	
ΦNsa =	19.7
ΦNcb =	16.2
ΦNpng =	31.6
Φnc =	N/A
ΦNna =	16.2

SHEAR KIPS			
ΦVsa =	10.2		
ΦVcb(g)+y =	4.80	ΦVcb(g)+x =	10.81
ΦVcb(g) -y =	34.36	ΦVcb(g) -x =	10.81
ΦcpVcp =	32.4		
ΦVn+y =	4.80	ΦVn+x =	10.22
ΦVn -y =	10.22	ΦVn- x =	10.22

HORIZONTAL LONGITUDINAL

SPACING & EDGE DISTANCES:

SPACING:

Minimum center to center spacing of untorqued cast-in-place anchors = 4do= **2.5** IN.

Minimum o.c. spacing of torqued cast-in-place & post-installed anchors = 6do= **3.8** IN.

EDGE DISTANCE:

Minimum edge distance for cast-in headed anchors, un-torqued shall be based on minimum cover requirements for reinforcement of ACI 7.7:

Concrete cast against and permanently exposed to soil: **3** IN.

Concrete exposed to earth or weather:
#6-#18 bars **2** IN.
#5 bar & smaller: **1 1/2** IN.

Concrete not exposed to weather or in contact with ground:
#14 & #18 bars: **1 1/2** IN.
#11 bars & smaller: **3/4** IN.

Minimum edge distance for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in ACI 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 test information, the minimum edge distance shall be taken as not less than:

Undercut anchors, 6do= **3.75** IN.

Torqued-controlled anchors, 8do= **5** IN.

Displacement - controlled anchors, 10do= **6.25** IN.

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Subject: ANCHOR DESIGN ACI 318-05 APPENDIX D
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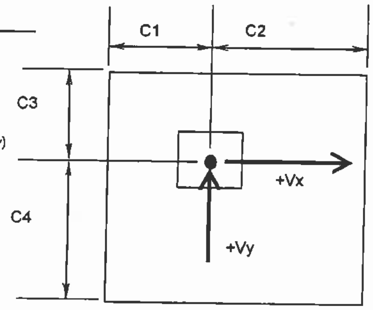
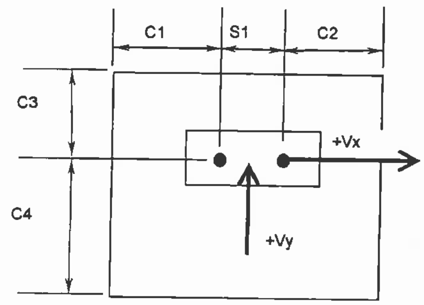
INPUT: Description: Portal Frame Group

No. of Anchors = 4 | C1 = 100 IN. | S1 = 4 IN.
C2 = 100 IN. | S2 = 4 IN.
Anchor Diameter = 0.750 IN. | C3 = 12 IN.
C4 = 100 IN.
ANCHOR STEEL DESIGNATION = A307 (A307, A325, A490, or leave blank for ACI design only)
fya = 36,000 YIELD STRENGTH (PSI) PER ASTM F 1554
futa = 58,000 TENSILE STRENGTH (PSI) PER ASTM F 1554
Anchor Embed. = 8 IN. (hef)

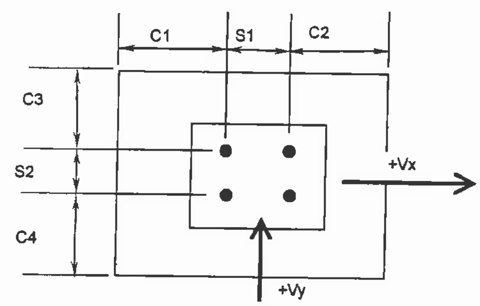
Anchor Type = CI (CI = Cast-In place , PI = Post Installed)
Ductile? DUCTILE DUCTILE or BRITTLE?
Built-Up Grout Pad = NO (YES or NO)
Anchor Head: H (H = headed anchor, J = J or L bolt)
Concrete : NW (NW = Normal weight, LW = All Light weight Concrete, SLW = Sand Light weight Concrete)
f'c = 3,000 PSI (28 Day Compressive Strength)
UNCRACKEI CRACKED or UNCRACKED Concrete
h = 24 IN, (Thickness of Concrete Member)
Condition: B A or B, ("A" applies when supplementary reinf. Is provided, per D.4.4)
Condition for P.I. Anchors = ________ (1, 2, or 3)



ONE ANCHOR



TWO ANCHORS



FOUR ANCHORS

TENSION:

Steel Strength: Nsa = n Ase futa | Φns = 0.75

]I 318 0.0 Ase = 0.334 IN^2 | futa = 1.9 fya or 125,000 PSI = 58,000 PSI
ΦNsa = 58.1 KIPS
AISC 7-14 ΦNs 59.7 KIPS | ΦFt = 33.8 KSI, AISC TABLE 7-14

ΦNsa = 58.1 Kips/GROUP	ΦNsa = 14.5 Kips/BOLT

Concrete Break-out Strength : Φcb = 0.70 | NW?= 1

ANco = 576.00 IN^2 (D-6) | Anc = 784.0 IN.(

Nb = 29,744.5 (LB) ACI D-7 & D-8

If three or four edge distances are less than 1.5 hef, (Ca,max<= 1.5 hef) then embedment is limited to the greater of Cmax/1.5, (hef = Ca,max/1.5) or 1/3 of the max spacing between anchors For use in eq. D-4 thru D-11
hef = 8.000 IN.

Ψec,N = 1.00 NO ECCENTRICITY | e'nx = 0
e'ny = 0

Ψed,N = 1.00 Modification for edge effects

Ψc,N = 1.25 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψc,N = 1.25 for cast-in anchors
Ψc,N = 1.4 for Post-installed anchors
Ψc,N = 1.0 Cracking, post-installed, or cast-in anchors

ΦNcb = 35.42 KIPS

(D-4, D-5)

Pullout Strength in Tension: Φpn = 0.70

Abrg = 0.911 | eh= 4.5do for J or L bolts

Np = 21,864.00 LB, D-15, D-16

Ψc,p = 1.4 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψ4 = 1.4 for cast-in anchors
Ψ4 = 1.0 Cracking, cast-in anchors

ΦNpn = 21.43 KIPS, Single Anchor (D-14)
ΦNpng = 85.71 KIPS, Group of Anchors (D-14)

Concrete side-face blowout strength of a headed anchor in tension:

Φnc = 0.7 | Not considered when Ca,min > 0.4 hef

Nsb= NOT CONSIDERED

C = 12 IN.
C2 = 100 IN.
C2/C = 3

C is equal to the minimum edge distance and C2 is equal to the minimum edge distance perpendicular to C.
If C2 < 3 C then multiply by an additional factor per D.5.4.2.

Φnc = N/A

SHEAR:

Steel Strength of Anchor in Shear: D6.1 Φvsa = 0.65

CI318 0.0, Ase = 0.334 IN² futa = 1.9 fya or 125,000 PSI = 58,000.0 PSI

Post-Installed Anchor Sleeves:
Asl = ______ IN2 futsl = ______ PSI

ΦVsa = 30.2 KIPS, D-18 or D-19
AISC 7-10 ΦVsa= 31.79 KIPS ΦFs = 18 KSI, AISC TABLE 7-10
Assume threads included in shear plane & single shear application

ΦVsa = 30.2 Kips/GROUP	ΦVsa = 7.6 Kips/BOLT

Concrete Breakout Strength of Anchor in Shear, per D.6.2: Φvs = 0.7

Avco+y =	648.0 IN²	Avco+x =	1152.0 IN²
Avco-y =	1152.0 IN²	Avco-x =	1152.0 IN²
Avc+y =	720.0 IN²	Avc+x =	960.0 IN²
Avc-y =	1248.0 IN²	Avc-x =	960.0 IN²
vb+y=	20920.8 LB	vb+x=	32209.7 LB
vb-y=	32209.7 LB	vb-x=	32209.7 LB
Ψec,v +y =	1.0	Ψec,v +x =	1.0
Ψec,v -y =	1.0	Ψec,v -x =	1.0
Ψed+y =	1.0	Ψed+x =	0.9
Ψed-y =	1.0	Ψed-x =	0.9
Ψc,v =	1.4		

Are the Anchors welded to the plate? NO
If yes, then anchor capacity is determined based on the row farthest from the edge and, the center to center spacing of the anchor is not less than 2.5 in.; and supplementary reinforcement is provided at the corners if C2 <= 1.5 hef

If three or more edge distances are less than 1.5 C1. (C2-4 or h<= 1.5 C1) then edge distance C1 is limited to C1 = h/1.5,
For use in eq. D-22 thru D-26

C'1+y =	12.0 IN	C'1+x =	16.0
C'1-Y =	16.0 IN	C'1-x =	16.0

e'vx = 0 IN., Eccentricity of shear force on a group of anchors, X axis
e'vy = 0 IN., Eccentricity of shear force on a group of anchors, Y axis

Ψc,v : D6.2.7, For anchors located in a region of concrete where analysis Indicates no cracking (ft < fr) at service loads, then Ψe,v = 1.4

Ψe,v = 1.0, Anchors in cracked concrete with no supplementary reinf.
Ψe,v = 1.2, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & the edge.
Ψe,v = 1.4, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & edge & #4 stirrups.

ΦVcb(g)+y =	22.78 KIPS	ΦVcb(g)+x =	22.36 KIPS
ΦVcb(g) -y =	34.20 KIPS	ΦVcb(g) -x =	22.36 KIPS

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR: D.6.3 : Φcp = 0.7

Ncb = 50.6 KIPS From D-4 & Above

ΦcpVcp = 70.8 KIPS

D-28

RESULTS:

TENSION KIPS	
ΦNsa =	58.1
ΦNcb =	35.4
ΦNpng =	85.7
Φnc =	N/A
ΦNna =	35.4

SHEAR KIPS			
ΦVsa =	30.2		
ΦVcb(g)+y =	22.78	ΦVcb(g)+x =	22.36
ΦVcb(g) -y =	34.20	ΦVcb(g) -x =	22.36
ΦcpVcp =	70.8		
ΦVn+y=	22.78	ΦVn+x=	22.36
ΦVn - y=	30.22	ΦVn- x=	22.36

HORIZONTAL LONGITUDINAL

↓

NO LOAD FROM PORTAL FRAMES

I-C OVE NTY CO #: 6 7-8

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Project:	Gila Country Sign Shop	Project Number:	30-09118-00
Subject:	ANCHOR DESIGN ACI 318-05 APPENDIX D	Computed by:	EDN
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SPACING & EDGE DISTANCES:

SPACING:

Minimum center to center spacing of untorqued cast-in-place anchors = 4do = **3.0** IN.

Minimum o.c. spacing of torqued cast-in-place & post-installed anchors = 6do = **4.5** IN.

EDGE DISTANCE:

Minimum edge distance for cast-in headed anchors, un-torqued shall be based on minimum cover requirements for reinforcement of ACI 7.7:

Concrete cast against and permanently exposed to soil: 3 IN.

Concrete exposed to earth or weather:
#6-#18 bars 2 IN.
#5 bar & smaller: 1 1/2 IN.

Concrete not exposed to weather or in contact with ground:
#14 & #18 bars: 1 1/2 IN.
#11 bars & smaller: 3/4 IN.

Minimum edge distance for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in ACI 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 test information, the minimum edge distance shall be taken as not less than:

Undercut anchors, 6do = 4.5 IN.

Torqued-controlled anchors, 8do = 6 IN.

Displacement - controlled anchors, 10do = 7.5 IN.

DLR Group

Calculations

Date
Project Name
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Computed
Checked
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BEAM LOADING DATA

Span		Channel	Max. Allowable Uniform Load		Deflection at Uniform Load		Uniform Loading at Deflections					
							Span/180		Span/240		Span/360	
In	mm		Lbs	kN	In	mm	Lbs	kN	Lbs	kN	Lbs	kN
24	610	P3000	1290	5.7	0.07	2	1290	5.7	1290	5.7	1290	5.7
		P3001	2660*	11.8	0.03	1	2660*	11.8	2660*	11.8	2660*	11.8
36	914	P3000	860	3.8	0.15	4	860	3.8	860	3.8	590	2.6
		P3001	2410	10.7	0.08	2	2410	10.7	2410	10.7	2410	10.7
48	1219	P3000	650	2.9	0.26	7	650	2.9	500	2.2	330	1.5
		P3001	1810	8.1	0.15	4	1810	8.1	1810	8.1	1620	7.2
60	1524	P3000	520	2.3	0.41	10	420	1.9	320	1.4	210	0.9
		P3001	1450	6.4	0.23	6	1450	6.4	1450	6.4	1040	4.6
72	1829	P3000	430	1.9	0.59	15	290	1.3	220	1.0	150	0.7
		P3001	1200	5.3	0.33	8	1200	5.3	1080	4.8	720	3.2
84	2134	P3000	370	1.6	0.80	20	220	1.0	160	0.7	110	0.5
		P3001	1030	4.6	0.45	12	1030	4.6	790	3.5	530	2.4
96	2438	P3000	320	1.4	1.03	26	170	85 point	120	0.5	80	0.4
		P3001	900	4.0	0.59	15	810	405 point	610	2.7	400	1.8
108	2743	P3000	290	1.3	1.33	34	130	0.6	100	0.4	70	0.3
		P3001	800	3.6	0.75	19	640	2.8	480	2.1	320	1.4
120	3048	P3000	260	1.2	1.64	42	110	0.5	80	0.4	50	0.2
		P3001	720	3.2	0.93	24	520	2.3	390	1.7	260	1.2
144	3658	P3000	220	1.0	2.40	61	70	0.3	60	0.3	40	0.2
		P3001	600	2.7	1.33	34	360	1.6	270	1.2	180	0.8
168	4267	P3000	180	0.8	3.11	79	50	0.2	40	0.2	30	0.1
		P3001	520	2.3	1.84	47	260	1.2	200	0.9	130	0.6
192	4877	P3000	160	0.7	4.13	105	40	0.2	30	0.1	NR	NR
		P3001	450	2.0	2.37	60	200	0.9	150	0.7	100	0.4
216	5486	P3000	140	0.6	5.15	131	NR	NR	NR	NR	NR	NR
		P3001	400	1.8	3.00	76	160	0.7	120	0.5	80	0.4
240	6096	P3000	130	0.6	6.56	167	NR	NR	NR	NR	NR	NR
		P3001	360	1.6	3.70	94	130	0.6	100	0.4	60	0.3

*Load limited by spot weld shear.

NR = Not Recommended

Notes:

1. Above loads include the weight of the member. This weight must be deducted to arrive at the net allowable load the beam will support.
2. Long span beams should be supported in such a manner as to prevent rotation and twist.
3. Allowable uniformly distributed loads are listed for various simple spans, that is, a beam on two supports. If load is concentrated at the center of the span, multiply load from the table by 0.5 and corresponding deflection by 0.8.
4. See page 66 for lateral bracing load reduction charts.

BEAM LOADING DATA

Span		Channel	Max. Allowable Uniform Load		Deflection at Uniform Load		Uniform Loading at Deflections					
							Span/180		Span/240		Span/360	
in	mm		Lbs	kN	in	mm	Lbs	kN	Lbs	kN	Lbs	kN
24	610	P5000 P5001	5260† 6170*†	23.4 27.4	0.03 0.01	1 0	5260 6170*†	23.4 27.4	5260 6170*†	23.4 27.4	5260 6170*†	23.4 27.4
36	914	P5000 P5001	3510 6170*†	15.6 27.4	0.07 0.02	2 1	3510 6170*†	15.6 27.4	3510 6170*†	15.6 27.4	3510 6170*†	15.6 27.4
48	1219	P5000 P5001	2630 5650†	11.7 25.1	0.12 0.05	3 1	2630 5650†	11.7 25.1	2630 5650†	11.7 25.1	2630 5650†	11.7 25.1
60	1524	P5000 P5001	2110 4520†	9.4 20.1	0.18 0.08	5 2	2110 4520†	9.4 20.1	2110 4520†	9.4 20.1	1920 4520†	8.5 20.1
72	1829	P5000 P5001	1750 3770	7.8 16.8	0.26 0.11	7 3	1750 3770	7.8 16.8	1750 3770	7.8 16.8	1330 3770	5.9 16.8
84	2134	P5000 P5001	1500 3230	6.7 14.4	0.36 0.15	9 4	1500 3230	6.7 14.4	1470 3230	6.5 14.4	980 3230	4.4 14.4
96	2438	P5000 P5001	1320 2830	5.9 12.6	0.47 0.20	12 5	1320 2830	660 POINT 1415 POINT	1130 2830	5.0 12.6	750 2830	3.3 12.6
108	2743	P5000 P5001	1170 2510	5.2 11.2	0.59 0.25	15 6	1170 2510	5.2 11.2	890 2510	4.0 11.2	590 2510	2.6 11.2
120	3048	P5000 P5001	1050 2260	4.7 10.1	0.73 0.31	19 8	960 2260	4.3 10.1	720 2260	3.2 10.1	480 2260	2.1 10.1
144	3658	P5000 P5001	880 1880	3.9 8.4	1.06 0.44	27 11	670 1880	3.0 8.4	500 1880	2.2 8.4	330 1690	1.5 7.5
168	4267	P5000 P5001	750 1610	3.3 7.2	1.43 0.60	36 15	490 1610	2.2 7.2	370 1610	1.6 7.2	250 1240	1.1 5.5
192	4877	P5000 P5001	660 1410	2.9 6.3	1.88 0.79	48 20	380 1410	1.7 6.3	280 1410	1.2 6.3	190 950	0.8 4.2
216	5486	P5000 P5001	580 1260	2.6 5.6	2.35 1.00	60 26	300 1260	1.3 5.6	220 1130	1.0 5.0	150 750	0.7 3.3
240	6096	P5000 P5001	530 1130	2.4 5.0	2.94 1.24	75 31	240 1130	1.1 5.0	180 910	0.8 4.0	120 610	0.5 2.7

*Load limited by spot weld shear. †Bearing load may govern capacity. See page 67.

Notes:
1. Above loads include the weight of the member. This weight must be deducted to arrive at the net allowable load the beam will support.
2. Long span beams should be supported in such a manner as to prevent rotation and twist.
3. Allowable uniformly distributed loads are listed for various simple spans, that is, a beam on two supports. If load is concentrated at the center of the span, multiply load from the table by 0.5 and corresponding deflection by 0.8.
4. See page 66 for lateral bracing load reduction charts.

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DATE:

Bentley DLR Group

MECHANICAL SUPPORT ANGLES
≤ 2000 LB, CENTERED

Current Date: 10/20/2009 3:37 PM
Units system: English
File name: \

Design Results

Continuous Beam
Design code ANSI/AISC 360-05 LRFD

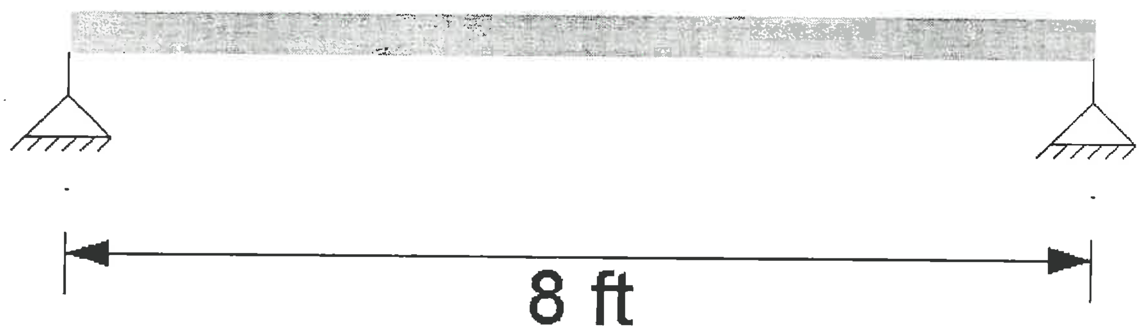
GENERAL INFORMATION:

Spans:

Span	Span length [ft]	Section	Material
1	8.00	T2LU 4X3X1_4LLBB	A36

Nodes:

Distance [ft]	Restraint	Tx	Ty	Rz
0.00	Pinned	1	1	0
8.00	Pinned	1	1	0



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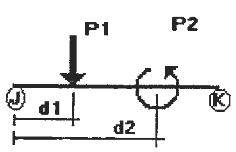
Load conditions:

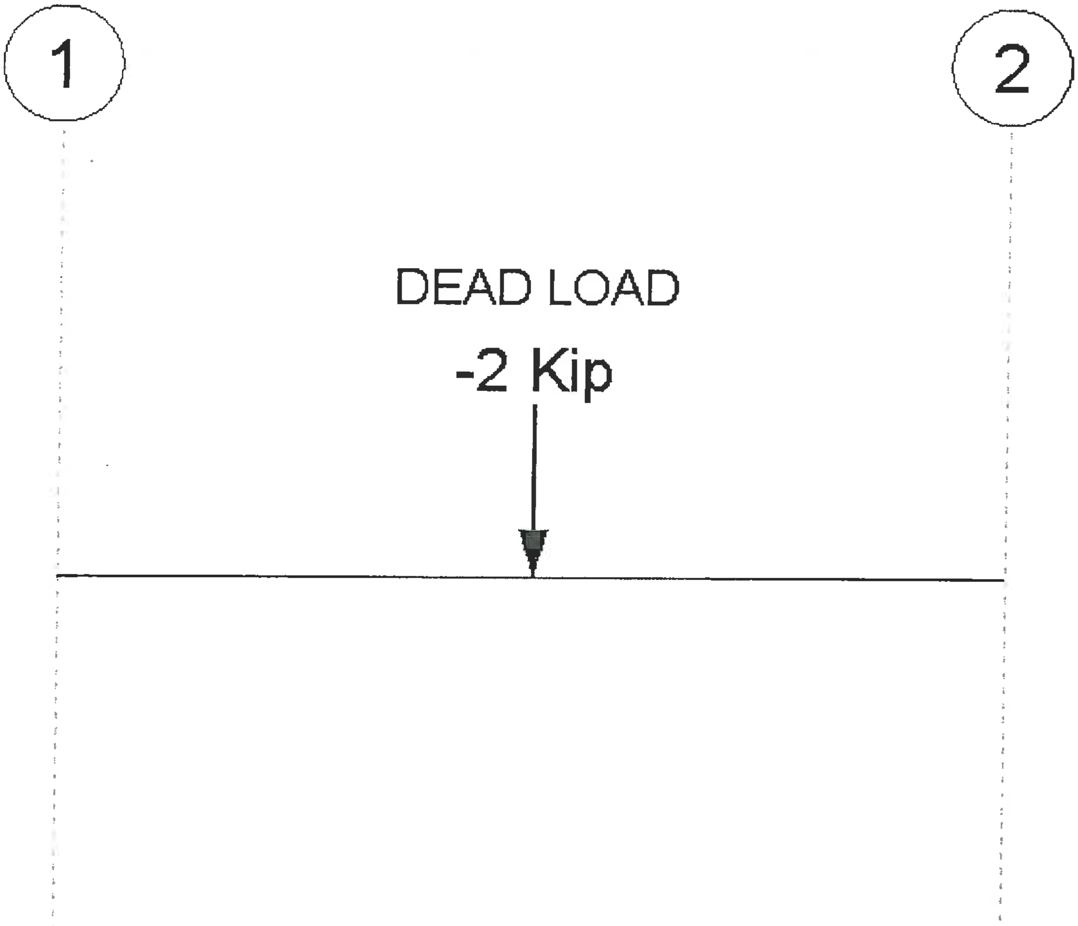
Condition	Description	Comb.	Category	Duration
DL	Dead Load	No	DL	--
LL	Live Load	No	LL	--
S1	DL+LL	Yes	Service	--
D1	1.4DL	Yes	Design	--
D2	1.2DL+1.6LL	Yes	Design	--

Concentrated forces and moments



Condition	Span	Dist [ft]	P [Kip]	M [Kip*ft]
DL	1	4.00	-2.00	0.00

Loads summary



Reactions:

Nodes	Load condition	Rx [Kip]	Ry [Kip]	Mz [Kip*ft]
1	D1	0.00	1.46	0.00
2	D1	0.00	1.46	0.00
1	D2	0.00	1.26	0.00
2	D2	0.00	1.26	0.00
1	Min.	0.00	1.26	0.00
2	Min.	0.00	1.26	0.00
1	Max.	0.00	1.46	0.00
2	Max.	0.00	1.46	0.00

Member forces and inflection points

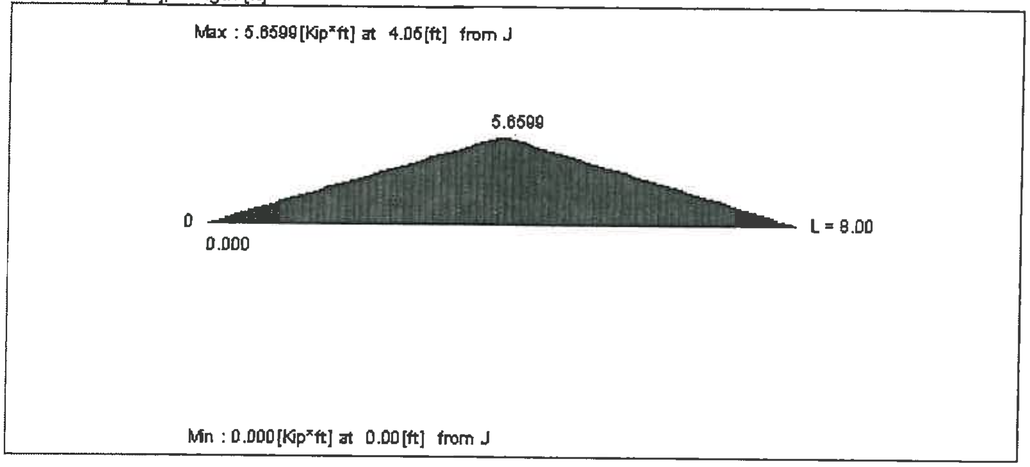
Station [%]	Condition	Distance [ft]	Shear V [Kip]	Moment M [Kip*ft]
0	D1	0.00	-1.46	0.00
50	D1	4.00	1.40	5.73
100	D1	8.00	1.46	0.00
0	D2	0.00	-1.26	0.00
50	D2	4.00	1.20	4.91
100	D2	8.00	1.26	0.00

Critical deflections

Condition	Span	Distance [ft]	@ [%]	Deflection [in]	Deflection f(L)	Allowable [in]
S1	1	4.00	50.00	0.23973	(L/400)	0.53333

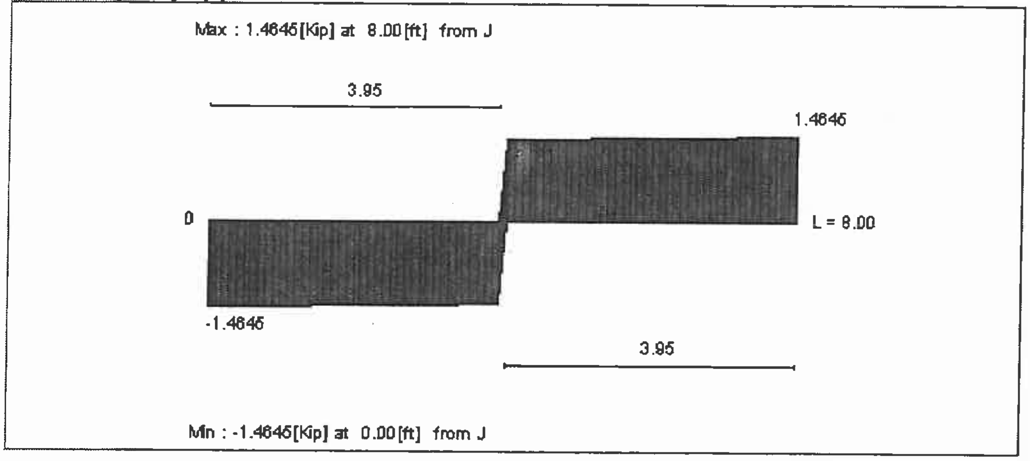
Envelopes :

M33 bending moment
Moments [Kip*ft], Length [ft]

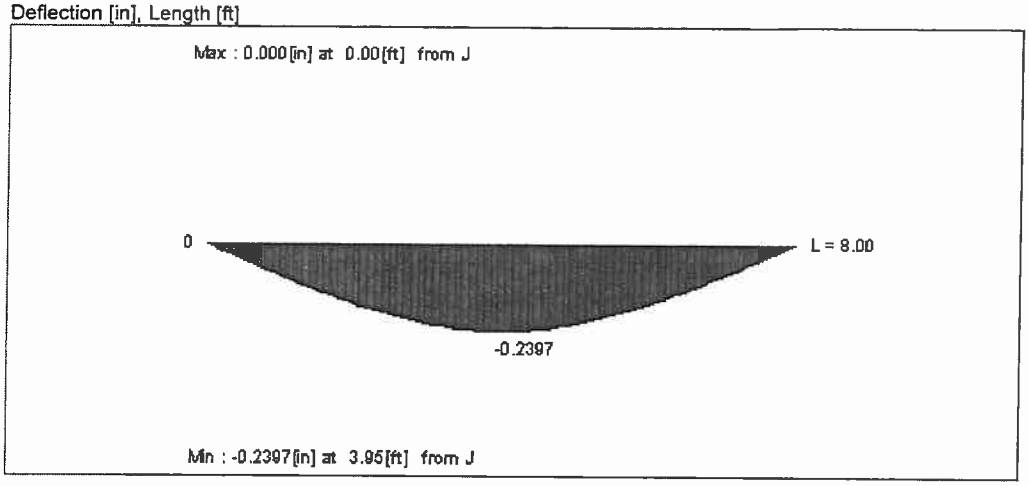


G6

V2 shear forces:
Forces [Kip], Length [ft]



Vertical Translation
Deflection [in], Length [ft]

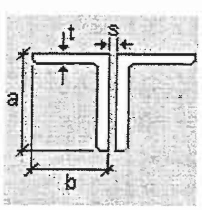


DESIGN:

Span	:	**1 (T2LU 4X3X1_4LLBB_A36)**
Design status	:	**OK**

PROPERTIES

Section : T2LU 4X3X1_4LLBB



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Height (a)	4.00	[in]
Width (b)	3.00	[in]
Separation (s)	0.00	[in]
Thickness (t)	0.25	[in]

Section properties	**Unit**	**Major axis**	**Minor axis**
Full unreduced cross-sectional area (A)	[in2]	3.38	
Moment of Inertia (local axes) (I)	[in4]	5.49	4.47
Moment of Inertia (principal axes) (I')	[in4]	5.49	4.47
Bending constant for moments (principal axis) (J')	[in]	-1.13	0.00
Radius of gyration (local axes) (r)	[in]	1.27	1.15
Radius of gyration (principal axes) (r')	[in]	1.27	1.15
Saint-Venant torsion constant (J)	[in4]	0.07	
Warping constant of the cross-section (Cw)	[in6]	0.15	
Distance from centroid to shear center (principal axis) (xo, yo)	[in]	0.00	-0.76
Top elastic section modulus of the section (local axis) (S top)	[in3]	4.50	1.51
Bottom elastic section modulus of the section (local axis) (S bot)	[in3]	1.98	1.51
Top elastic section modulus of the section (principal axis) (S' top)	[in3]	4.50	1.51
Bottom elastic section modulus of the section (principal axis) (S' bot)	[in3]	1.98	1.51
Plastic section modulus (local axis) (Z)	[in3]	3.55	2.49
Plastic section modulus (principal axis) (Z')	[in3]	3.55	2.49
Polar radius of gyration (ro)	[in]	2.03	
Area for shear (Aw)	[in2]	1.50	2.00
Torsional modulus (1/C)	--	3.56	

Material : A36

Properties	**Unit**	**Value**
Yield stress (Fy):	[Kip/in2]	36.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11507.94

DESIGN CRITERIA

Description	**Unit**	**Major axis**	**Minor axis**
Top unbraced length between lateral supports (LbTop)	[ft]	8.00	
Bottom unbraced length between lateral supports (LbBop)	[ft]	8.00	
Effective length factor (K)	–	1.00	1.00
Effective length factor for torsion	–	1.00	
Length for axial tension (L)	[ft]	8.00	
Unbraced compression length (Lx, Ly)	[ft]	8.00	8.00
Length for torsion and lateral-torsional buckling	[ft]	8.00	
Clear distance between longitudinal connectors	[ft]	0.00	
Additional hypotheses			
Continuous lateral torsional restraint		No	
Tension field action		No	

SERVICE CONDITIONS

Verification	**Unit**	**Value**	**Ctrl EQ**	**Reference**
Tension				
Maximum geometric slenderness (L/r)	–	83.48		(Sec. D1)
Compression				
Geometric critical slenderness (KL/r)	–	83.48		(Sec. E2)

Compression and flexure

Deflection	[in]	-0.24	S1 at 50.00%

DESIGN CHECKS

DESIGN FOR FLEXURE (ϕ= 0.90) ✔

Bending about major axis, M33

Ratio	:	0.67			
Capacity	:	8.53 [Kip*ft]	Ctrl Eq.	:	D1 at 50.00%
Demand	:	5.73 [Kip*ft]	Reference	:	(Sec. F)

Intermediate results	Unit	Value	Reference
Yielding (Mp)	[Kip*ft]	9.48	(Sec. F)
Lateral-torsional buckling (LTB Mn)	[Kip*ft]	56.49	(Sec. F)
Modification factor for lateral-torsional buckling (Cb)	--	1.31	(Sec. F1)
Lateral-torsional factor (c)	--	1.00	(Sec. F2.2)
Factor for lateral-torsional buckling in tees and 2L (B)	--	0.76	(Eq. F9-5)
Web local buckling (WLB Mn)	--	N/A	(Sec. F)
Local buckling (LB Mn)	--	N/A	(Sec. F)
Flange local buckling (FLB Mn)	--	N/A	(Sec. F)
Slenderness parameter for flange (λ)	--	12.00	(Sec. B4)
Limiting slenderness parameter for compact flange (λp)	--	15.33	(Sec. B4)
Limiting slenderness parameter for noncompact flange (λr)	--	25.83	(Sec. B4)
Tension flange yielding (TFY Mn)	--	N/A	(Sec. F)

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	6.72 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%
Demand	:	0.00 [Kip*ft]	Reference	:	(Sec. F)

Intermediate results	Unit	Value	Reference
Yielding (Mp)	[Kip*ft]	7.46	(Sec. F)
Flange local buckling (FLB Mn)	--	N/A	(Sec. F)
Slenderness parameter for flange (λ)	--	12.00	(Sec. B4)
Limiting slenderness parameter for compact flange (λp)	--	15.33	(Sec. B4)
Limiting slenderness parameter for noncompact flange (λr)	--	25.83	(Sec. B4)

DESIGN FOR SHEAR ✔

Shear parallel to major axis, V3 (ϕ= 0.90)

Ratio	:	0.00			
Capacity	:	29.16 [Kip]	Ctrl Eq.	:	D1 at 0.00%
Demand	:	0.00 [Kip]	Reference	:	(Sec. G)

Intermediate results	Unit	Value	Reference
Web Shear coefficient (Cv)	--	1.00	
Web plate buckling coefficient (kv)	--	1.20	(Sec. G2)

Shear parallel to minor axis, V2 (ϕ= 0.90)

Ratio	:	0.04			
Capacity	:	38.88 [Kip]	Ctrl Eq.	:	D1 at 0.00%
Demand	:	-1.46 [Kip]	Reference	:	(Sec. G)

Intermediate results	Unit	Value	Reference
Web Shear coefficient (Cv)	--	1.00	
Web plate buckling coefficient (kv)	--	1.20	(Sec. G2)

DESIGN FOR TENSION (ϕ= 0.90) ✔

Tension

Ratio	:	0.00			
Capacity	:	109.51 [Kip]	Ctrl Eq.	:	D1 at 0.00%
Demand	:	0.00 [Kip]	Reference	:	(Sec. D)

DESIGN FOR COMPRESSION (ϕ= 0.90) ✔

Compression

Ratio	:	0.00			
Capacity	:	63.75 [Kip]	Ctrl Eq.	:	D1 at 0.00%
Demand	:	0.00 [Kip]	Reference	:	(Sec. E)

Intermediate results	Unit	Value	Reference
Slenderness parameter for web (λw)	--	16.00	(Sec. B4)
Limiting slenderness parameter for noncompact web (λrw)	--	15.89	(Sec. B4)
Slenderness parameter for flange (λf)	--	12.00	(Sec. B4)
Limiting slenderness parameter for noncompact flange (λrf)	--	12.77	(Sec. B4)
Elastic flexural stress (Fex)	[Kip/in2]	50.44	(Eq. E4-9)
Elastic flexural stress (Fey)	[Kip/in2]	41.07	(Ec. E4-10)
Elastic torsional buckling stress (Fez)	[Kip/in2]	58.09	(Eq. E4-11)
Critical elastic flexural-torsional buckling stress (Fe)	[Kip/in2]	30.63	(Sec.E4)
Critical flexural buckling stress (Fcr)	[Kip/in2]	23.49	(Sec.E)
Critical flexural-torsional buckling stress (FcrTor)	[Kip/in2]	20.96	(Sec.E4)
Stress reduction factor in unstiffened elements (Qs)	--	0.91	(Sec.E7)
Effective section reduction factor in stiffened elements (Qa)	--	1.00	(Sec.E7)
Effective area at a uniform stress (Aeff)	[in2]	3.38	(Sec.E7)

DESIGN FOR TORSION (ϕ= 0.90) ✔

Torsion

Ratio	:	0.00			
Capacity	:	0.46 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%
Demand	:	0.00 [Kip*ft]	Reference	:	(Sec. H3)

Intermediate results	Unit	Value	Reference
Critical stress (Fcr)	[Kip/in2]	21.60	(Sec. H)

INTERACTION ✔

Combined axial and flexure interaction value

Ratio	:	0.67			
Ctrl Eq	:	D1 at 50.00%	Reference	:	(H1-1b)

Combined shear and torsion interaction value

Ratio	:	0.04			
Ctrl Eq.	:	D1 at 0.00%	Reference	:	(Ec. 4.9) DG 9

CRITICAL STRENGTH RATIO ✔

Ratio	:	0.67			
Ctrl Eq	:	D1 at 50.00%	Reference	:	(Sec. F)

H1

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2001 North American Specification w/2004 Supplement ASD
DATE: 10/20/2009

SECTION DESIGNATION: 800S137-54 Single

INPUT PROPERTIES:

Web Height =	8.000 in		Steel Thickness =	0.0566	in
Top Flange =	1.375 in		Inside Corner Radius =	0.0849	in
Bottom Flange =	1.375 in		Yield Stress, Fy =	33.0	ksi
Stiffening Lip =	0.375 in		Fy With Cold-Work, Fya =	33.0	ksi
Punchout Width =	1.500 in		Punchout Length =	4.000	in

Ceiling Solver Design Data - Simple Span

Joist Span 24.00 ft

Deflection Limit L/360

Joist Spacing 24.0 in

Dead Load = 7.0 psf

DL Multiplied by 1.00 for Strength Checks

Check Flexure

Flexural Bracing: KyLy = 48.0 in

Cb = 1.00

Me = 4350 Ft-Lb

My = 3513 Ft-Lb

0.56 My < Me < 2.78 My

Mc = 3028 Ft-Lb

Sc/Sf = 0.94

Mmax = 1008 Ft-Lb <= Ma = 1704 Ft-Lb

Check Deflection

Deflection Limit: L/360

Maximum Deflection = 0.698 in

Deflection Ratio = L/413

Check Shear

Vmax = 168 lb (Including Flexural Load Multiplier)

Shear capacity reduced for punchouts

Va = 2091 lb >= Vmax

Check Web Crippling

Rmax = 168 lb (Including Flexural Load Multiplier)

Web Crippling capacity reduced for punchouts

End Bearing Length = 1.00 in

Ra = 359 lb >= Rmax, stiffeners not required

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APPROVED FIELD
GILA COUNTY COMMUNITY DE
PERMIT #: 60911-01
DATE: 7-8-10 BY:

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2001 North American Specification w/2004 Supplement ASD
DATE: 10/20/2009

SECTION DESIGNATION: 800S137-43 Single

INPUT PROPERTIES:

Web Height =	8.000	in	Steel Thickness =	0.0451	in
Top Flange =	1.375	in	Inside Corner Radius =	0.0712	in
Bottom Flange =	1.375	in	Yield Stress, Fy =	33.0	ksi
Stiffening Lip =	0.375	in	Fy With Cold-Work, Fya =	33.0	ksi
Punchout Width =	1.500	in	Punchout Length =	4.000	in

Ceiling Solver Design Data - Simple Span

Joist Span 24.00 ft
Deflection Limit L/240
Joist Spacing 24.0 in
Dead Load = 7.0 psf
DL Multiplied by 1.00 for Strength Checks

Check Flexure

Flexural Bracing: KyLy = 48.0 in
Cb = 1.00
Me = 3566 Ft-Lb
My = 2842 Ft-Lb
0.56 My < Me < 2.78 My
Mc = 2459 Ft-Lb
Sc/Sf = 0.88
Mmax = 1008 Ft-Lb <= Ma = 1301 Ft-Lb

Check Deflection

Deflection Limit: L/240
Maximum Deflection = 0.886 in
Deflection Ratio = L/325

Check Shear

Vmax = 168 lb (Including Flexural Load Multiplier)
Shear capacity reduced for punchouts
Va = 1051 lb >= Vmax

Check Web Crippling

Rmax = 168 lb (Including Flexural Load Multiplier)
Web Crippling capacity reduced for punchouts
End Bearing Length = 1.00 in
Ra = 234 lb >= Rmax, stiffeners not required

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